SOILS ENGINEERING, INC.

GEOTECHNICAL INVESTIGATION
SEWAGE FEASIBILITY STUDY
FOR THE PROPOSED DOLLAR GENERAL STORE TO BE LOCATED ON HIGHWAY 138 NORTHWEST OF OASIS ROAD IN PINON HILLS, SAN BERNARDINO COUNTY, CALIFORNIA

Prepared for:

SimonCRE, Via Soleri II, LLC
5111 N. Scottsdale Road, Suite 200
Scottsdale, Arizona
85250-7009

SEI File No. 15-15400

July 15, 2015

REGISTERED PROFESSIONAL ENGINEER
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July 15, 2015

SEI File No. 15-15400

SIMONCRE Via Soleri II, LLC
5111 N. Scottsdale Road, Suite 200
Scottsdale, AZ 85250-7009

Attention: Mr. David Friedberg

Subject: Geotechnical Investigation & Sewage Feasibility Study
Project: Proposed Dollar General Store
Location: Hwy 138, NW of Oasis Road, Pinon Hills, San Bernardino County, CA

Dear Mr. Friedberg:

In accordance with your request, we have performed a Geotechnical Investigation and Sewage Feasibility Study at the subject site. Recommendations for site preparation and grading, and criteria for foundation design are provided in the attached report.

Appendix A, "Guide Specifications for Earthwork," is provided as a supplement to Section I, "Earthwork," in the recommendations of the report.

Appendix B, "Field Investigation," contains Logs of Test Borings, Figures 2 through 6, and a site plan, Figure 1, showing approximate locations of test borings.

Appendix C, "Soils Test Data," contains tabulations of laboratory test data.

Appendix D, "Seismic Investigation," contains information provided by EQFAULT & UBCSEIS.

We hope this provides the information you require. If you have any questions regarding the contents of our report, or if we can be of further assistance, please contact us.

Respectfully submitted,
SOILS ENGINEERING, INC.

TONY M. FRANGIE
No. 39549

Tony Frangie, P.E.
Vice President

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GEOTECHNICAL INVESTIGATION
SEWAGE DISPOSAL FEASIBILITY STUDY
FOR THE PROPOSED DOLLAR GENERAL STORE
TO BE LOCATED ON HIGHWAY 138
NW OF OASIS ROAD
IN
PINON HILLS, SAN BERNARDINO COUNTY, CALIFORNIA

SCOPE

This report was prepared to provide recommendations for preparation and grading, and criteria for selection and design of foundation for the proposed structures. The following recommendations are addressed herein:

EARTHWORK

Site preparation and grading in areas to receive the proposed structure(s) and pavements.

Quality control of engineered fill.

FOUNDATIONS

Foundation types most adequate for the proposed structures.

Anticipated total and differential settlements.

Lateral earth pressures for designing retaining walls and for evaluating the passive and frictional resistance of foundations.

PAVEMENT

Structural section design recommendations for proposed roadways.

SITE LOCATION AND CONDITIONS

The proposed improvements are located north west of Oasis Road on Highway 138 in Pinon Hills, San Bernardino County, California. The site has ungraded natural surfaces with a few dirt trails traversing the lot. The property is bounded by residential on the west and south, where smoke tree road passes this site on the south. On the east and north commercial buildings sparsely line highway 138. A drainage basin passes this property along the northern property line.

GEOLOGIC SETTING

The site area has a gentle slope to the north. The project site rests on thousands of feet of alluvial sediments identified as Quaternary alluvium Deposits (Qoa) on geologic maps within the eastern portion of the Mojave Desert. The closest active fault is the San Andreas Fault located approximately 8.3 kilometers to the south. The Cucamonga Fault is located approximately 16.7 kilometers to the south. The Cleghorn Fault is located approximately 22.0 Kilometers to the south. The Sierra Madre Fault zone is located approximately 23.2 kilometers to the west. The San Jacinto fault is located approximately 25.2 kilometers to the southeast. Other major faults within 35 miles include; the Clamshell (27.1 km), San Jose (34.4 km), North Frontal (37.2 km), Raymond (42.5km) and the Whittier (55.9 km). The site is not located in an Alquist-Priolo (AP) California Earthquake Fault Zone.

SUBSURFACE CONDITIONS

Earth materials encountered during our field investigation consisted of surface soils of light yellowish brown to yellowish brown, fine, dry, stiff to very stiff, sandy silt and yellowish brown to light yellowish brown fine, dry to damp silty sand. These surface layers were underlain by light yellowish brown, fine, medium dense, dry poorly-graded sand. These soils are described as ML, SM and SP, respectively, in the Unified Soils Classification System (USCS). The near surface soils are in a stiff to very stiff condition for the sandy silt and loose to medium dense condition for the silty sand. They should provide adequate support for the proposed structures provided that a portion of the surface soils are excavated and compacted as outlined in the earthwork recommendations of this report. Detailed descriptions of the various soils encountered during our field investigation are shown on Figures 2 through 5 in Appendix B, "Field Investigation." A "Key to Symbols" legend describing the symbols in the boring logs is also attached.

GROUNDWATER CONDITIONS

No free groundwater was encountered in any of our test borings to the maximum depth tested. Moreover, groundwater should be deep enough to be of no concern to foundation stability.

SEWAGE DISPOSAL FEASIBILITY

Subsurface conditions and ground surface topography are conducive to the construction of functional on-site sewage disposal systems consisting of a septic tank or leach fields. Percolation tests were performed using the "Manual of Septic-Tank Practice" issued by the Public Health Service of the U.S. Department of Health, Education, and Welfare. Test results are provided in Table 1, in Appendix B. Percolation test locations are shown on the Boring Location Map, Figure 1.

Leach fields should be designed for UPC Soil Type 4 and constructed in substantial accordance with the requirements of the Kern County Health Department.

SEISMIC DESIGN VALUES

The seismic design values are presented in the table below based on the 2013 California Building Code (CBC). The Site Class for the proposed improvements located 0.2 miles northwest of Oasis Road on Highway 138 in Pinon Hills, San Bernardino County, California were determined using standard penetration test data obtained at the site and are provided in the attached Boring Logs.

SEISMIC DESIGN CRITERIA	VALUE	SOURCE
Risk Category	**II**	2013 CBC Table 1604.5
Site Class	**D**	Site Specific Soils Report 2013 CBC Section 1613.3.2, ASCE 7-10 Table 20.3-1
Mapped MCE_R Spectral Response Acceleration, short period, S_s	**1.640**	USGS maps/Software - 2013 CBC Figure 1613.3 (1)
Mapped MCE_R Spectral Response Acceleration, at 1-sec. Period, S_1	**0.775**	USGS Maps/Software - 2013 CBC Figure 1613.3 (2)
Site Coefficient, F_a	**1.000**	USGS Software - 2013 CBC Table 1613.3.3 (1)
Site Coefficient, F_v	**1.500**	USGS Software - 2013 CBC Table 1613.3.3 (2)
Adjusted MCE_R Spectral Response Acceleration, Short periods, $S_{MS} = F_a S_s$	**1.640**	USGS Software - 2013 CBC Section 1613.3.3
Adjusted MCE_R Spectral Response Acceleration, 1-sec. Period, $S_{M1} = F_v S_1$	**1.163**	USGS Software - 2013 CBC Section 1613.3.3
Design Spectral Response Acceleration, short periods, $S_{DS} = 2/3 S_{MS}$	**1.093**	USGS Software - 2013 CBC Section 1613.3.4
Design Spectral Response Acceleration, 1-sec period, $S_{D1} = 2/3 S_{MI}$	**0.775**	USGS Software - 2013 CBC Section 1613.3.4
Peak Ground Acceleration (PGA) for Max.	***0.658g***	*USGS Software – ASCE 7-10 Fig 22-7*

SEISMIC DESIGN CRITERIA	VALUE	SOURCE
Considered Earthquake (MCE_G)		
Site Coefficient, F_{PGA} = 1.00, $PGA_M = F_{PGA} * PGA$ =	***0.658g***	*USGS Software – ASCE 7-10 Table 11.8-1*
Site-Specific Ground Motion Procedures for Seismic Design, C_{RS}	***1.002***	*USGS Software - ASCE 7-10 Fig 22-17*
Site-Specific Ground Motion Procedures for Seismic Design, C_{R1}	***0.957***	*USGS Software - ASCE 7-10 Fig 22-18*
Seismic Design Category short periods (S_{DS})	**D**	2013 CBC Table 1613.3.5 (1)
Seismic Design Category, 1-sec period (S_{D1})	**D**	2013 CBC Table 1613.3.5 (2)

MCE_R = Maximum Considered Earthquake (risk targeted),
MCE_G = Maximum Considered Earthquake (geometric mean)

Site Seismic Parameters

Major fault systems and their distances from the site are given in EQFault Summary attached in Appendix D. The site is not located within an Alquist-Priolo Earthquake Fault Zone (AP Zone). The San Andreas Fault is located approximately 8.2 kilometers south of the site. The largest maximum site acceleration based on deterministic methods is 0.667g from a 8.0 magnitude earthquake on the San Andreas Fault approximately 8.2 kilometers away. See Appendix D for copies of the computer modeling data.

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RECOMMENDATIONS

I. EARTHWORK

"Earthwork Specifications," in Appendix A are provided for general guidance in preparing site grading plans. In addition, the following specific recommendations are provided and supersede the latter wherever discrepancies may exist:

A. Compaction

Unless otherwise specified herein, the terms, "Compaction," or "Compacted," wherever used or implied within this report should be interpreted as compaction to 90 percent of the maximum density obtainable by ASTM Test Method D1557.

B. Optimum Moisture

The term, "Optimum Moisture," wherever used or implied within this report, should be interpreted as that obtained by the above described test method.

C. Stripping

Prior to soil compaction, existing ground surfaces should be stripped of surface vegetation. A stripping depth of one inch should be adequate. In no instance should stripped material be used in engineered fill or blended with and compacted in original ground.

D. Ground Surface Preparation

Proposed Structure Areas:

Ground surfaces in the proposed building areas shall be compacted in accordance with the following procedures:

1. Excavate earth material to a minimum depth of two feet below the lowest grade in the proposed building area.
2. The bottom of the excavation shall be reviewed by the soil engineer or his representative prior to any backfill operations. The top foot (1) of materials exposed at the bottom of the excavation shall be scarified and compacted to a minimum of 90 percent of ASTM D1557.
3. Moisten excavated and imported soils to near the optimum moisture or to a moisture content consistent with effective compaction and soil stability. Compact moistened soils to a minimum of 90 percent of the maximum density obtained by ASTM Test Method D1557.

4. Work to lines at least five feet beyond the outside edges of exterior footings and two feet beyond pavement edges.

***Pavement Areas*:**

Ground surfaces to receive concrete driveway and bituminous pavements should be scarified and compacted to a minimum depth of 12 inches below the grading plane in cut areas or to 12 inches in areas to receive fill. Engineered fill placed in proposed pavement areas should conform to the requirements of Section 5.4, "Placing, Spreading and Compacting Fill Materials," of Appendix A.

Compaction in proposed pavement areas should be a minimum of 90 percent of the maximum density as obtained by ASTM Test Method D1557, and should extend to a minimum of two feet beyond the outside edges of pavements. *The top eight (8) inches of subgrade below the grading plane shall be compacted to a minimum of 95%.*

***Utility Lines*:**

Backfill for utility lines traversing areas proposed for facilities, pavements, concrete slabs-on-grade, or areas to receive engineered fill for future construction should be compacted in accordance with the same requirements for adjacent and/or overlying fill materials.

Compaction should include haunch area, spring line, and from top of pipe to finished subgrade. The haunch area up to one foot above the top of the pipe should be backfilled with "cohesionless" material.

Cohesionless native materials may be used for trench and pipe or conduit backfill. The term "cohesionless," as used herein, is defined as material which when dry, will flow readily in the haunch areas of the pipe trench.

Pipe backfill materials should not contain rocks larger than two inches in maximum dimension. Where adjacent native materials exposed on the trench bottoms contain protruding rock fragments larger than two inches in maximum dimension, conduits and pipelines should be laid on a bedding consisting of clean, cohesionless sand (SP), in the Unified Soils Classification System.

Compaction Requirements - where not otherwise specified in our plans or in these recommendations, the following compaction requirements are applicable to all electrical, gas or water conduits:

TABLE A
COMPACTION DEPTH

Area	Haunch to 1 ft. Above Top of Pipe	1 ft. Above Top of Pipe to 2'6" Below Finished Grade	2'6" Below Finished Grade to Finished Subgrade
Structures	90%	90%	90%
Pavements	90%	90%	90%*
Non-Structural	90%	90%	90%

* *The top eight (8) inches of subgrade in the pavement area shall be compacted to a minimum of 95% of ASTM D1557.*

E. Engineered Fill

Earth materials obtained on-site are acceptable for use as engineered fill provided that all grasses, weeds and other deleterious debris are first removed.

Engineered fill materials should be placed in thin layers (less than ten inches uncompacted thickness), brought to near the optimum moisture content or to a moisture content commensurate with effective compaction and soil stability, and compacted to a minimum of 90 percent of the maximum density obtainable by ASTM Test Method D1557, "Placing, Spreading and Compacting Fill Materials," in Appendix A.

F. Imported Fill

The table shown below provides general guidelines for acceptance of import engineered fill. Materials of equal or better quality than on-site material could be reviewed by the Geotechnical Engineer on a case-by-case basis. No soil materials shall be imported onto the project site without prior approval by the Geotechnical Engineer. Any deviation from the specifications given below shall be approved by the Geotechnical Engineer prior to import operations.

Maximum Percent Passing #200 Sieve	40
Maximum Percent Retained 3" Sieve	0
Maximum Percent Retained 1½" Sieve *for building areas*	15
Maximum Percent Retained ¾" Sieve *for landscape areas*	5
Maximum Plasticity Index	26
Minimum R-Value	50

Furthermore, the soils proposed for import shall be generally homogenous and shall not contain cemented or clayey and/or silty lumps larger than one inch. When such lumps are present, they shall not represent more than ten percent (10%) of the material by dry weight. Where a proposed import source contains obviously variable soils, such as clay and/or silt layers, the soils which do not meet the above requirements shall be segregated and not used for this project or the various layers shall be thoroughly mixed prior to acceptance testing by the Geotechnical Engineer. The contractor shall provide sufficient advance notice, prior to import operations, to allow testing and evaluation of the proposed import materials. Because of the time needed to perform the above tests, the contractor shall provide a means by which the Geotechnical Engineer or others can verify that the soil(s) which was sampled and tested is the same soil(s) which is being imported to the project.

G. Drainage

Finished ground grades adjacent to the proposed buildings should be sloped to provide positive free drainage away from the foundations. No areas should be constructed that would allow drainage generated on the site, or water impinging upon the site from outside sources, to pond near footings and slabs or behind curbs.

Where ground surfaces adjacent to subsurface walls are to be landscaped, walls should be waterproofed. Installation of gravel-filled drains to route subsurface drainage away from walls will reduce the thickness of damp-proofing resulting in a considerable savings.

H. Slopes

Both fill and cut slopes should be constructed at 2:1 (horizontal to vertical) in accordance with the 2013 Uniform Building Code.

Finished slopes nearer than five feet from building foundations should be graded no steeper than five horizontal to one vertical (5:1). A slope ratio of two horizontal to one vertical (2:1) should provide adequate stability for slopes farther than five feet from footing lines.

The fill slopes shall be compacted to a minimum of 90% of ASTM D-1557 and in accordance with the Guide specifications for Earthwork, Appendix A. This may be achieved by overfilling the constructed slope and trimming to a compacted finished surface, rolling the slope face with a sheepsfoot as the level of the fill is raised, or any method that achieves the desired product.

The cut portion of the slope should be constructed first. Prior to construction of the fill slope, incompetent surface soils should be removed from the top of the cut.

Areas to receive fill or to support structures, slabs or pavements should be removed of all vegetation, debris and disturbed soils. All existing uncertified fill soils should be excavated to expose competent native soils.

Existing underground pipelines, private sewage disposal systems and any water or oil wells, if encountered during grading, should be removed or capped in accordance with procedures considered acceptable by the appropriate governing agency. Tree roots to 2 inches in diameter should be removed.

Both fill and cut slopes will be subject to erosion immediately after grading, and should be designed to reduce surficial sloughing by implementing a permanent slope maintenance program as soon as practical after completion of slope construction.

Slope maintenance should include proper care of erosion and drainage control devices, rodent control, and immediate planting with deep-rooting, lightweight, drought-resistant vegetation. An erosion control geotextile, may also be used in combination with vegetation to control erosion.

Experience has shown that slope performance is largely dependent upon proper slope maintenance (i.e., planting, proper watering, clearing of drainage devices, etc.). Slopes properly placed and conscientiously maintained are not expected to display excessive raveling or sloughing.

II. FOUNDATIONS

The proposed structure can be adequately supported on either continuous or isolated reinforced concrete spread footings designed in accordance with the criteria given below in Table B.

TABLE B
FOUNDATION DESIGN CRITERIA

Footing Type	Minimum Width (ft.)	Minimum Depth Below Lowest Adjacent Subgrade (ft.)	Maximum Allowable Soil Bearing Pressure (lbs./sq.ft.)
Continuous	1	1	2000
Isolated	1	1	2000

Bearing pressures given are for the minimum widths and depths shown above.

Bearing pressures given above are for dead and sustained (loads acting most of the time) live loads; they may be increased by one-third for wind and/or seismic loading conditions.

Settlement:

Provided maximum allowable soil bearing pressures given above are not exceeded, total settlement should not exceed one inch for a span of twenty feet. A major portion ... two-thirds to one-half ... of total settlement should occur before the end of construction. Differential settlements should occur before the end of construction. Differential settlements should, accordingly, be less than one-half of an inch for a span of twenty feet.

III. MODULUS OF SUBGRADE REACTION

Modulus of subgrade reaction for use in design of foundations is based on ranges of values for soil types provided by Foundation Analysis and Design by Joseph E Bowles.[1] Equation 1 should be used for footings on sandy soils. Foundations on clay soils should employ Equation 2. Equation 3 is for rectangular footings having dimensions w= **b** (width) and l = **mb** (length) the variable "m" being the ratio of the length to the width of the foundation. **K_s1** is the modulus of subgrade reaction from the source referenced above based on a 1 foot x 1 foot square plate. For general guidance K_s1 of 280 kcf may be used.

Equation (1) $$k_{sf} = K_{s1} \times \left(\frac{B+1}{2B}\right)^2$$

Equation (2) $$k_{sf} = K_{s1} \times B$$

Equation (3) $$k_{sf} = K_{s1} \times \frac{m+.5}{1.5m}$$

Values given above should be used for guidance. Local values may be higher or lower and should be based on results of in-situ plate bearing tests performed in accordance with ASTM Test Method D1194.

IV. LATERAL EARTH PRESSURES

Lateral earth pressures and friction coefficients for determining the passive lateral resistance of foundations against lateral movement and the active lateral forces against retaining walls and subsurface walls, expressed as equivalent fluid pressures, are given below in Table C. Lateral earth pressures were computed assuming that backfill materials are essentially free draining and level; and that no surcharge loads or sloping backfills are present within a distance from the wall equal to or less than the height (H)* of the wall.

(H)* = The height of backfill above the lowest adjacent ground surface.

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[1] Bowles, Joseph E; FOUNDATION ANALYSIS AND DESIGN; McGraw-Hill Book Company (1977); Table 9-1 pg 269

TABLE C
LATERAL EARTH PRESSURES

Case	Lateral Earth Pressures
Active	32 P.C.F.
Passive	350 P.C.F.
At-Rest	40 P.C.F.

Active Case: Active lateral earth pressures should be used when computing forces against free standing retaining walls, unrestrained at their tops. Active pressures should not be used where tilting outward of the walls greater than .002H would not be desirable.

Passive Case: Passive lateral earth pressures should be used when computing the lateral resistance provided by undisturbed or compacted native soils against the movement of footing. When computing passive resistance, the upper one foot of embedment depth should be discounted.

At-Rest Case: At-rest pressures should be used for subsurface walls restrained at their tops by floor diaphragms or tie-backs and for retaining walls where tilting outward greater than .002 H would not be desirable.

Frictional Resistance: A friction coefficient of **0.37** may be used when computing the frictional resistance of footings, grade beams, and slabs-on-grade to sliding. Frictional resistance and passive lateral soil resistance may be combined without reduction.

V. SOIL CORROSIVITY

Soluble Sulfates (SO_4)

The highest Sulfate (SO_4) concentration measured was 60 ppm. Generally, sulfate concentrations greater than 1,500 ppm are considered to be corrosive to foundation elements. (Ref: ACI 318, Section 4.3, Table 4.3.1)

Chlorides (Cl)

The highest Chloride (Cl) concentration measured was 6.6 ppm. Generally, chloride concentrations greater than 500 ppm are considered to be corrosive to foundation elements. (Ref: Caltrans Corrosion Guidelines / Version 1.0)

pH

The soil pH was 7.21. Generally, a pH level less than 5.5 are considered to be corrosive to foundation elements. (Ref: Caltrans Corrosion Guidelines/ Version 1.0)

Although preliminary test results indicate that soil corrosivity at the locations and depths tested is low to negligible, we anticipate that site grading operations will result in a blend of native and/or imported materials at finished subgrade elevations. Accordingly, additional tests should be performed after rough grading has been completed and prior to concrete design.

VI. SLABS-ON-GROUND

Slabs-on-ground may be supported on earth materials prepared in accordance with the recommendations of this report. If expansive soils are present, the slab and foundations may require special design in order to reduce potential damage from soil volume changes.

We recommend that moisture protection be provided for those proposed interior slabs-on-ground on which moisture sensitive floor coverings, structural elements, or equipment are to be installed or on which moisture sensitive items are to be stored. The project designer should provide specific details regarding construction of the concrete slab-on-ground, including the moisture barrier (if required) or vapor retarder, capillary break (if included), and blotter material (if included). The American Concrete Institute recommends a minimum vapor retarder of 10 mil thick polyethylene. Prior to construction of the slab, the vapor retarder should be protected from damage. Punctures and tears should be repaired prior to concrete placement.

If a blotter layer between the moisture vapor retarder/barrier and the concrete is to be used, it should consist of crusher fines or sand with predominantly angular, inter-locking grains that could be compacted with 100% of the material passing the #4 sieve screen. The material should, at the time of concrete placement, be dry to damp, compact, and smooth. Concrete should not be placed if the blotter layer is wet, or may become wet due to weather condition, as it will act as a water reservoir beneath the concrete and all apparent advantages of its use will be nullified. For slabs which are to be water-cured, the blotter layer, which would act as a reservoir, is not recommended. For further consideration, refer to American Concrete Institute *Manual of Concrete Practice 302.1R and 360.*

Pressurized water lines should not be placed beneath slabs. Gravity flow sewer lines may underlie slabs, but they should be routed to exit by the shortest feasible path.

VII. PAVEMENT

We have prepared a pavement design for the project area. The design is based on the lowest R-Value test performed on samples retrieved from the site and a Traffic Index (TI) ranging from 5 to 9. Pavement Design is provided in the tables provided on Pages 14 and 15.

PAVEMENT DESIGN SECTION TI OF 5.0

Subgrade R-Value	Class II Aggregate Base R-Value	Traffic Index	Hot Mix Asphalt (ft)	Aggregate Base (ft)	Compacted Native Subgrade* (ft)
64	78 min	5.0	0.20'	0.35'	1.0'

PAVEMENT DESIGN SECTION TI OF 6.0

Subgrade R-Value	Class II Aggregate Base R-Value	Traffic Index	Hot Mix Asphalt (ft)	Aggregate Base (ft)	Compacted Native Subgrade* (ft)
64	78 min	6.0	0.25'	0.35'	1.0'

PAVEMENT DESIGN SECTION TI OF 7.0

Subgrade R-Value	Class II Aggregate Base R-Value	Traffic Index	Hot Mix Asphalt (ft)	Aggregate Base (ft)	Compacted Native Subgrade* (ft)
64	78 min	7.0	0.35'	0.35'	1.0'

PAVEMENT DESIGN SECTION TI OF 8.0

Subgrade R-Value	Class II Aggregate Base R-Value	Traffic Index	Hot Mix Asphalt (ft)	Aggregate Base (ft)	Compacted Native Subgrade* (ft)
64	78 min	8.0	0.40'	0.35'	1.0'

PAVEMENT DESIGN SECTION TI OF 9.0

Subgrade R-Value	Class II Aggregate Base R-Value	Traffic Index	Hot Mix Asphalt (ft)	Aggregate Base (ft)	Compacted Native Subgrade* (ft)
64	78 min	9.0	0.45’	0.35’	1.0’

**The subgrade compaction shall be according to our recommendations provided in Section I, Earthwork, Paving Areas, of this report.*

These recommendations are valid only if the pavement is properly drained and shoulder areas are graded to prevent water ponding at pavement edges. All construction should be subject to adequate tests and observations to verify conformance with these recommendations.

VIII. LIMITATIONS, OBSERVATIONS AND TESTING

Conclusions and recommendations in this report are given for the proposed Dollar General Store to be located on Hwy 138, Northwest of Oasis Road in Pinon Hills, San Bernardino County, CA and are based on the following:

a. The information retrieved from five exploratory borings drilled at the subject site to a maximum depth of 16½ feet below the existing ground surface.

b. Our laboratory testing program results.

c. Our engineering analysis based on the information defined in this report.

d. Our experience in the San Bernardino County area.

Variations in soil type, strength and consistency may exist between specific boring locations. These variations may not become evident until after the start of construction. If such variations appear, a re-evaluation of the soils test data and recommendations may be necessary.

Unless a Geotechnical Engineer of this firm is afforded the opportunity to review plans and specifications, we accept no responsibility for compliance with design concepts or interpretations made by others with regard to foundation support, fill selection, fill placement or other recommendations presented in this report.

Changes in conditions of the subject property can occur with time because of natural processes or the works of man on the subject site or on adjacent properties. Changes in applicable engineering and construction standards can also occur as the result of legislation or from the broadening of knowledge. Accordingly, the finding of this report may be invalidated, wholly or in part, by changes beyond our control. Therefore, this report is subject to review and should not be relied upon without review after a period of two years or after any modifications to the site.

Review of Earthwork Operations

Review of earthwork operations relating to site clearing, ground stabilization, placement and compaction of fill materials, and finished grading is critical to the structural integrity of building foundation and floor systems. While the preliminary Geotechnical investigation and report provide guidelines which are used by the design team, i.e., architects, grading engineers, structural engineers, landscape engineers, etc., in completing their respective tasks, review of plans and site review and testing during earthwork operations are vital adjuncts to the completion of the Geotechnical engineer's tasks.

The most prevalent cause of failure of a structure foundation system is lack of adequate review and testing during the earthwork phase of the project. Projects rarely reach completion without some alteration being required such as may result from a change in subsurface conditions, an amendment in the size and scope of the project, a revision of the grading plans or a variation in structural details. Occasionally, even minor changes can significantly affect the performance of foundations.

The most prevalent secondary cause for foundation failure is inadequate implementation of Geotechnical recommendations during the formulation of foundation designs and grading plans. The error in a foundation design or an omission of a key element from a grading plan occurs most often as a result of inadequate communication between the various project consultants and -- when a change in consultants occurs -- improper transfer of authority and responsibility[2].

It is imperative, therefore, that any revisions to the project scope, any change in structural detail, or change in consultant, be brought to the attention of Soils Engineering, Inc. to allow for timely review and revision of recommendations and for an orderly transfer of responsibility and approval.

It is the responsibility of the owner or his representative to ensure that a representative of our firm is present at all times during earthwork operations relating to site preparation and grading, so that relative compaction tests can be performed, earthwork operations can be observed and compliance with the recommendations provided herein can be established.

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[2] If the civil engineer, the soils engineer, the engineering geologist or the testing agency of record is changed during the course of the work, the work shall be stopped until the replacement has agreed to accept the responsibility within the area of his technical competence for approval upon completion of the work.

This engineering report has been prepared within the limits prescribed to us by the client or his representative, in accordance with the generally accepted principles and practices of Geotechnical engineering. No other warranty, expressed or implied, is included or intended in this report.

Respectfully submitted,
SOILS ENGINEERING, INC.

APPENDIX A

GENERAL GUIDE SPECIFICATIONS FOR EARTHWORK

1. GENERAL

1.1 Scope

These specifications and plans include all earthwork pertaining to site rough grading including, but not limited to furnishing all labor and equipment necessary for clearing and grubbing; stripping; preparation of ground surfaces to receive fill; excavation; placement and compaction of structural and non-structural fill; disposal of excess materials and products of clearing, grubbing, and stripping; and any other work necessary to bring ground elevations to the lines and grades shown on the project plans.

1.2 Performance:

It shall be the responsibility of the contractor to complete all earthwork in accordance with project plans and specifications. No variance from plans and specifications shall be permitted without written approval of the Engineer-of-Record, hereinafter referred to as the "engineer" or his designated representative, hereinafter referred to as the "soils engineer." Earthwork shall not be considered complete until the "engineer" has issued a written statement confirming substantial compliance of earthwork operations to these specifications and to the project plans.

The contractor shall assume sole responsibility for job site conditions during the course of earthwork operations on the project, including safety of all persons and preservation of all property; this requirement shall apply continuously and not be limited to normal working hours. The contractor shall defend, indemnify, and hold harmless the owners, engineer, and soils engineer from any and all liability and claims, real or alleged, arising out of performance of earthwork on this project, except from liability incurred through sole negligence of the owner, engineers, or soils engineers.

2. DEFINITIONS

2.1 Excavations:

Excavation shall be defined within the content of these specifications as earth material excavated for the purpose of constructing fill embankment; grading the site to elevations shown on project plans; or placing underground pipelines, conduits, or other subsurface utilities or minor structures.

Excavations shall be made true to the lines shown on project plans and to within plus or minus one-tenth (0.1) of a foot, of grades shown on the accepted site grading plans.

2.2 Engineered Fill:

Engineered fill shall be construed within the body of these specifications as earth materials conforming to specifications provided in the soils or geotechnical report placed to raise the grade of the site, to backfill excavations, or to construct asphaltic concrete or Portland cement concrete pavement; and upon which the soils engineer has performed sufficient tests and has made sufficient observation during placement and compaction to enable him to issue a written statement confirming substantial conformance of the work to project earthwork specifications.

2.3 On-Site Material:

On-site material is earth material obtained in excavation made on the project site.

2.4 Imported Material:

Imported materials are earth materials obtained off the site, hauled in, and placed as fill.

2.5 "Compaction" or "Compacted:"

Wherever expressed or implied within the context of these specifications shall be interpreted as compaction to ninety (90) percent of the maximum density obtainable by ASTM Test Method D1557.

2.6 Grading Plane:

The grading Plane is the surface of the basement material upon which the lowest layer of subbase, base, asphaltic or Portland cement concrete, surfacing, or other specified layer is placed.

3. SITE CONDITIONS

The contractor shall visit the site, prior to bid submittal, to determine existing soil and topographic conditions, and the nature of materials that may be encountered during the course of the work under this contract, and make his own interpretation of the contents of the Geotechnical Report, as they pertain to said conditions.

The contractor shall assume all liability under the contract for any loss sustained as a result of variations which may exist between specific soil boring locations or changed conditions resulting from natural or man-made circumstances occurring after the date of the Preliminary Field Investigations.

4. CLEARING AND GRUBBING

4.1 Clearing and Grubbing

Clearing and grubbing shall consist of removing all debris such as metal, broken concrete, trash, vegetation growth and other biodegradable substances, from all areas to be graded. Existing obstructions below shall be removed in accordance with the following procedures:

4.1.1 **Slabs and Pavements** - Shall be completely removed. Asphaltic or Portland Cement, concrete fragments may be used in engineered fills provided they are broken down to a maximum dimension of six (6.0) inches and thoroughly dispersed within a friable soil matrix. Engineered fill containing said fragments should not be placed above the elevation of the bottom of the lowest structure footing.

4.1.2 **Foundations** - existing at the time of grading shall be removed to a depth not less than two (2.0) feet below the bottom of the lowest structure footing.

4.1.3 **Basements, Septic Tanks** – buried concrete containers of similar construction located within areas destined to receive pavements, structures, or engineered fills should be completely removed and disposed of off the site. Basements, septic tanks, etc., situated outside structures, or structural fill areas shall be disposed of by breaking an opening in bottoms to permit drainage, and by breaking walls down to not less than two (2.0) feet below finished subgrade.

4.1.4 **Buried Utilities** – such as sewer, water and gas lines or electrical conduits to remain in service shall be re-routed to pass no closer than four (4.0) feet to the outside edge of proposed exterior footings of structures. Lines to be abandoned shall be completely removed to a minimum depth of two (2.0) feet below finished building pad grade. Concrete lines deeper than two (2.0) feet below finished building pad grade and having diameters less than six (6.0) inches can be crushed in place.

4.1.5 **Root Systems** – shall be completely removed to a minimum depth of two (2.0) feet below the bottom of the lowest proposed structure footing or to two (2.0) feet below finished subgrade, whichever depth is lower. Root systems deeper than the elevation indicated above shall be excavated to allow no roots larger than two (2.0) inches in diameter.

4.1.6 **Cavities** – resulting from clearing and grubbing or cavities existing on the site as a result of man-made or natural activity shall be backfilled with earth materials placed and compacted in accordance with Sections 5.3 and 5.4 of these specifications.

4.1.7 **Preservation or Monuments, Construction Stakes, Property Corner Stakes**, or other temporary or permanent horizontal or vertical control reference points shall be the responsibility of the contractor. Where these markers are disturbed, they shall be replaced at the contractor's expense.

5. SITE GRADING

Site grading shall consist of excavation and placement of fills to lines and grades shown on the project plans and in accordance with project specifications and recommendations of the Preliminary Soils Report, whichever is more stringent. The following are recommendations issued in this report.

5.1 Areas to Receive Fill:

5.1.1 Surfaces to receive fill shall be scarified to a depth of at least six (6.0) inches, or as recommended in this report, whichever is greater, until the surface is free from ruts, hummocks or other uneven features which would tend to prevent uniform compaction by the equipment to be used.

5.1.2 After the area to receive fill has been cleared and scarified, it shall be moistened and compacted to a depth of at least six (6.0) inches in accordance with specifications for compacting fill material in paragraph 5.4, below.

5.2 Excavation:

5.2.1 Excavations shall be cut to elevations plus or minus 0.1 foot of the grades shown on the accepted plans.

5.2.2 When excavated materials are to be used in engineered fill, the excavation shall be made in a manner to produce as much mixing of the excavated materials as practicable.

5.2.3 When excavations are to backfilled, and where surfaces exposed by excavation are to support structures or concrete floor slabs, the exposed surfaces shall be scarified, moistened and compacted, as stated above for areas to receive fill. Over excavation below specified depths will not eliminate the requirement for exposed surface compaction.

5.3 Fill Materials:

5.3.1 Materials obtained from on-site excavations will be considered satisfactory for construction of on-site engineered fills unless otherwise stated in the Soils Report or Foundation Investigation. If unexpected pockets of poor or weak materials are encountered in excavations, and they cannot be upgraded by mixing with other materials or by other means, they may be rejected by the soils engineer for use in engineered fill. Rocks larger than 12 inches in size in any dimension shall not be allowed in the proposed building area. If a large amount of rocks greater than 12 inches in size in any dimension is encountered a rock disposal area shall be located on the grading plan. Rocks shall be mixed with well graded soils to assure that the voids in these areas will fill properly.

5.3.2 When imported fill materials are necessary to bring the site up to planned grades, no material shall be imported prior to its approval and acceptance by the soils engineer.

5.3.3 The soils engineer shall be given notice of the proposed source of imported materials with adequate time allowance for his testing of the proposed materials. The time required for testing will vary with different types of materials, job conditions, and ultimate function of filled areas. Under best conditions the time requirement will not be less than 48 hours.

5.4 Placing, Spreading, and Compacting Fill Material:

5.4.1 The fill materials shall be placed in layers which, when compacted, shall not exceed six (6.0) inches in thickness. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to insure uniformity of material in each layer. Increased thickness of layers may be approved by the soils engineer when conditions warrant.

5.4.2 All fills shall be placed in level layers; layers shall be continuous over the area of any structural unit, and all portions of the fill shall be brought up simultaneously within the area of any structural unit. When imported material is used, it must be placed so that its thickness is as uniform as possible within the area of any structural unit.

5.4.3 When materials are to be excavated and replaced in a compacted condition, segmented, or leap-frogging of cut-fill operation within the area of any structural unit will not be permitted unless the method is specifically described by the soils engineer.

5.4.4 When the moisture content of fill material is below the lower limit specified by the Soils Engineer, water shall be added until the moisture content is as specified; and when it is above the upper limit specified, the material shall be aerated by blading or other satisfactory methods until the moisture content is as specified.

5.4.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted to not less than ninety (90) percent of maximum density in accordance with ASTM Density Test Method D1557. Compaction shall be by equipment of such design that it will be able to compact the fill to specified density. When the soils engineer specifies a specific type of compaction equipment to be used, such equipment shall be used as specified.

5.4.6 Compaction of each layer shall be continuous over its entire area and the equipment shall make sufficient trips to insure that the desired density has been obtained.

5.4.7 Field density tests shall be made by the soils engineer. The compaction of each layer of fill shall be subject to testing. Where sheepsfoot rollers are used, the soil may be disturbed to a depth of several inches. Density tests shall be taken in the compacted material below the disturbed surface. When tests indicate the density of any layer of fill or portion thereof is below the required ninety (90) percent density, the particular layer or portion shall be re-worked until the required density has been obtained.

5.4.8 When the soils engineer specifies compaction to other standards or to percentages other than ninety (90) percent, such specification, with respect to the particular items shall supersede these specifications.

5.4.9 The fill operation shall be continued in six (6) inch compacted layers, as specified above, until the fill has been brought to within 0.1 foot, plus or minus of the finished slopes and grades, as shown on the accepted plans. The finished surface of fill areas shall be graded or bladed to a smooth and uniform surface and no loose material shall be left on the surface.

5.4.10 No fill materials shall be placed, spread, or compacted while it is frozen or thawing or during unfavorable weather conditions. When work is interrupted by weather conditions, fill operations shall not be resumed until the soils engineer indicates that moisture content and density of previously placed fill are satisfactory.

5.5 Observations and Testing:

The soils engineer shall be provided with a 48 hour advance notice, in order that he may be present at the site during all earthwork activities related to excavation, tree root removal, stripping, backfill, and compaction and filling of the site and to perform periodic compaction tests so that substantial conformance to these recommendations can be established.

APPENDIX B

FIELD INVESTIGATION

Five (5) test borings were drilled at the subject site and terminated at a maximum depth of 16.5 feet below the existing ground surface. Borings were advanced using an eight (8.0) inch hollow-stem auger. Test data and descriptions from these holes form the basis of the conclusions and recommendations contained in this report.

Undisturbed samples and disturbed bulk samples were obtained. Undisturbed samples were taken using either a 2-3/8" (inside diameter) split-barrel sampler or a 1-3/8" (inside diameter), 2" (outside diameter) Standard Penetration Sampler (SPT). Penetration resistance of undisturbed soils was obtained by driving the above described sampler using a one-hundred-forty pound hammer falling a distance of thirty (30.0) inches and recording blow counts for each six (6.0) inch increment of drive on Test Boring Logs. In addition, bulk soil samples, selected as most representative of near surface soils encountered, were taken for laboratory testing.

As drilling progressed, earth materials encountered were logged and classified in accordance with the Unified Soils Classification System and presented graphically on Logs of Test Borings, Figures 2 through 6, along with the Legend.

Approximate locations of test borings are shown on the boring Location Map, Figure 1.

Two Percolation tests were performed in the field in the area of Borings 4 and 5. Results of these tests are shown on Table 1, Percolation Table, attached in Appendix C.

UNIFIED SOIL CLASSIFICATION SYSTEM
(USCS)

COARSE-GRAINED SOILS
Less than 50% Fines*

Group Symbols	Description	Major Divisions
GW	Well-graded gravels or gravel-sand mixtures, less than 5% fines	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size
GP	Poorly-Graded gravels or gravel-sand mixture less than 5% fines	
GM	Silty Gravels, Gravel-sand silt mixtures, more than 12% fines	
GC	Clayey Gravels, gravel-sand-clay mixtures, more than 12% fines	
SW	Well-Graded sands or Gravelly Sands, less than 5%	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
SP	Poorly-graded Sands or Gravelly Sands, less than 5% fines	
SM	Silty Sands, Sand-Silt Mixtures, more than 12% fines	
SC	Clayey Sands, Sand-Clay Mixtures, more than 12% fines	

NOTE: coarse-grained soils receive dual symbols if they contain 5 to 12% fines (e.g. SW-SM, GP-GC, etc.)

FINE-GRAINED SOILS
More than 50% Fines

Group Symbols	Description	Major Divisions
ML	Inorganic Silts, very fine sands, Rock Flour, Silty or Clayey Fine Sands	SILTS AND CLAYS Liquid Limit less than 50
CL	Inorganic Clays of low to medium plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays	
OL	Organic Silts or Organic Silt-Clays of Low Plasticity	
MH	Inorganic Silts, Micaceous or Diatomaceous Fine Sands or Silts, Elastic Silts	SILTS AND CLAYS Liquid limit more than 50
CH	Inorganic Clays of High Plasticity, Fat clays	
OH	Organic Clays of Medium to High Plasticity	
PT	Peat, Mulch, and other Highly Organic Soils	HIGHLY ORGANIC SOILS

NOTE: Fine-grained soils may receive dual classification based upon plasticity characteristics

SOIL SIZES

COMPONENT	SIZE RANGE
BOULDERS	ABOVE 12 in.
COBBLES	3 in. to 12 in.
GRAVEL Coarse Fine	No. 4 to 3 in. 3/4 in. to 3 in. No. 4 to 3/4 in.
SAND Coarse Medium Fine	No. 200 to No. 4 No. 10 to No. 4 No. 40 to No. 10 No. 200 to No. 40
* Fines (Silt or Clay)	BELOW No. 200

NOTE: Only sizes smaller than three inches are used to classify soils

CONSISTENCY

CLAYS & SILTS	BLOWS/FOOT*
VERY SOFT	0 – 2
SOFT	2 – 4
FIRM	4 – 8
STIFF	8 – 15
VERY STIFF	15 – 30
HARD	Over 30

RELATIVE DENSITY

SANDS & GRAVELS	BLOWS/FOOT*
VERY LOOSE	0 – 4
LOOSE	4 – 10
MEDIUM DENSE	10 – 30
DENSE	30 – 50
VERY DENSE	Over 50

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1 3/8" I.D.) Split-spoon (ASTM D1586)

PLASTICITY OF FINE GRAINED SOILS

PLASTICITY INDEX	VOLUME CHANGE POTENTIAL
0 - 15	Probably Low
15 - 30	Probably Moderate
30 or more	Probably High

DRY: no feel of moisture
DAMP: much less than normal moisture
MOIST: normal moisture
WET: much greater than normal moisture
SATURATED: at or near saturation

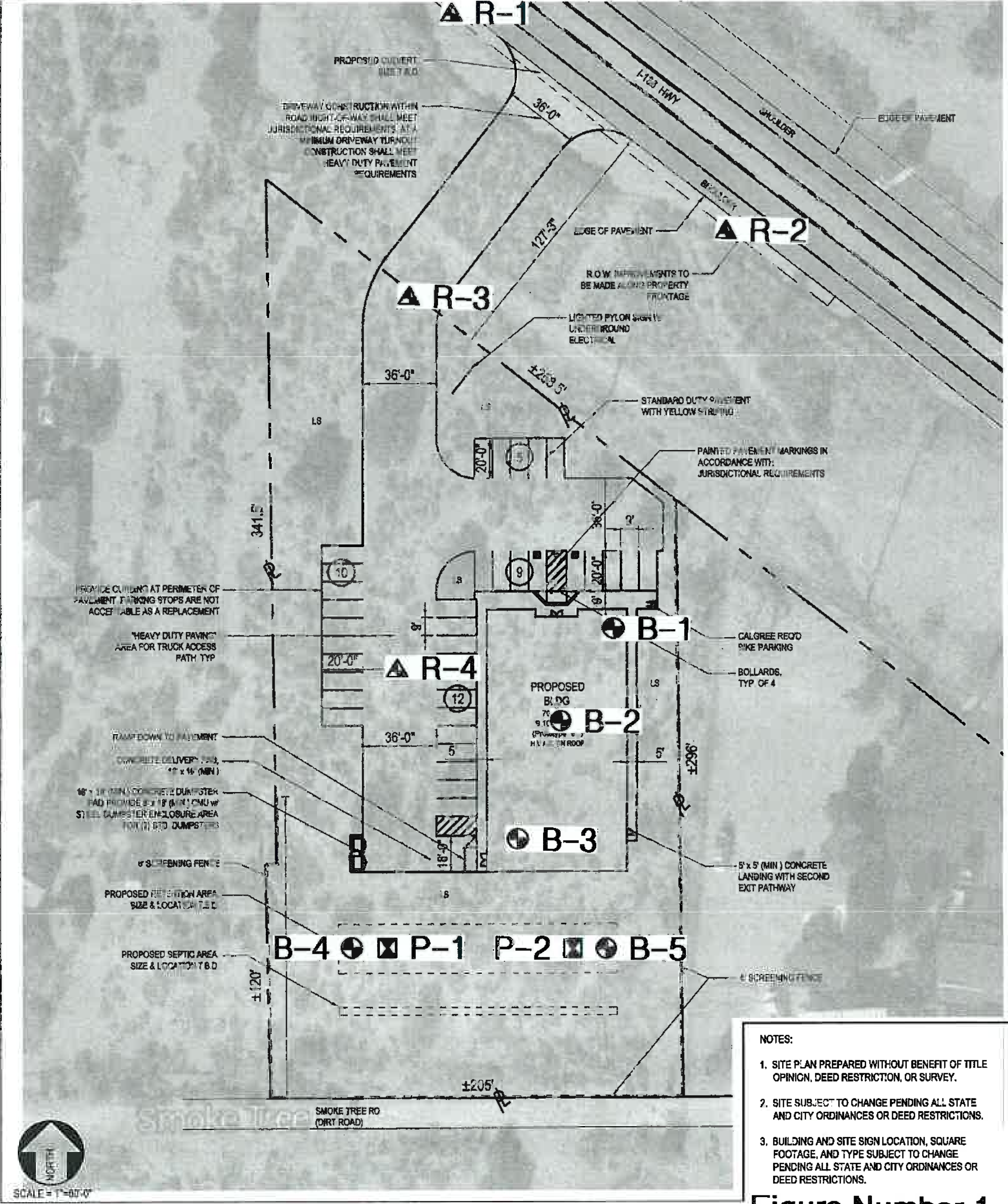
Soils Engineering Inc.

PRELIMINARY SITE PLAN

CITY, STATE - STREET:
PINON HILLS, CA - HIGHWAY 138 NEAR MOUNTAIN RD.

			DEVELOPER		DESIGNER		DATE:
PROTOTYPE:	B						
BLDG/SALES SF:	9,026/7,220	COMPANY:	SIMON CRE	COMPANY:	MPA ARCHITECTS, INC.		05-13-15
ACREAGE:	±1.73 GROSS	NAME:	JOSHUA SIMON	NAME:	LEONARDO DALE		
REQ'D PARKING SPACES: (1 SPACE/250 SQ.FT.)	36	PHONE #:	480-745-1956	PHONE #:	619-236-0595		

SOILS ENGINEERING, INC.
SEI File No. 15-15400



NOTES:

1. SITE PLAN PREPARED WITHOUT BENEFIT OF TITLE OPINION, DEED RESTRICTION, OR SURVEY.
2. SITE SUBJECT TO CHANGE PENDING ALL STATE AND CITY ORDINANCES OR DEED RESTRICTIONS.
3. BUILDING AND SITE SIGN LOCATION, SQUARE FOOTAGE, AND TYPE SUBJECT TO CHANGE PENDING ALL STATE AND CITY ORDINANCES OR DEED RESTRICTIONS.

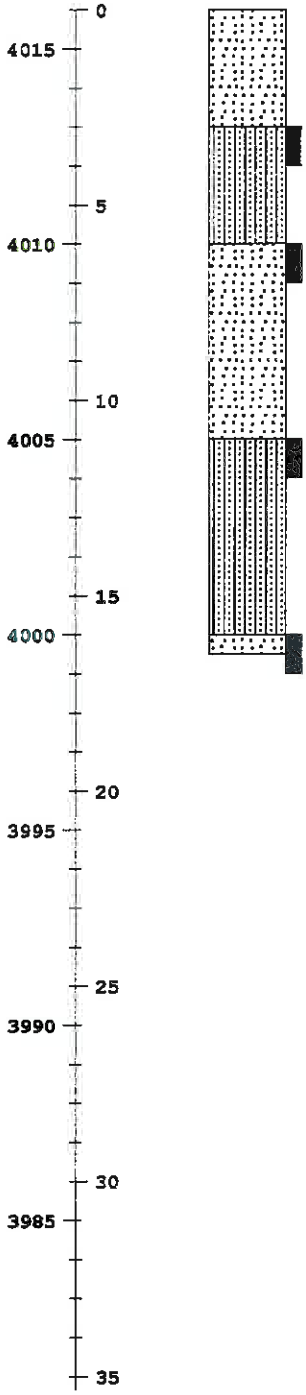
Figure Number 1

LOG OF TEST BORING
BORING B-1

Page 1 of 1

PROJECT: *PINON HILLS DOLLAR GENERAL STORE*
BORING DATE: *6/25/2105*
BORING LOCATION: *SEE BORING LOCATION MAP, FIGURE 1*
DRILL METHOD: *4-1/4 INCH I.D. HOLLOW-STEM AUGER*
DESCRIPTION: *GEOTECHNICAL INVESTIGATION*
DEPTH TO WATER - ▽ **:** *N/A*
CAVING - ➤ **:** *N/A*

FILE NO: *15-15400*
ELEV.: *4016'*
START: *6/25/2015*
FINISH: *6/25/2015*
LOGGER: *MIKE WATTS*



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Remarks	Density pcf	Moisture %
		SP	POORLY-GRADED SAND: light yellowish brown; dry; fine to medium grained; rock; trace silt.			
	9/6 11/6 11/6	SM	SILTY SAND: light yellowish brown; dry; poorly-graded; slightly cohesive; medium dense.		113.2	1.5
	11/6 14/6 26/6	SP	POORLY-GRADED SAND: light yellowish brown; dry; non-cohesive; rock; dense.		121.1	0.8
	7/6 13/6 20/6	SM	SILTY SAND: light yellowish brown; dry; slightly cohesive; dense; trace gravel.		112.4	1.1
	8/6 17/6 24/6	SP	POORLY-GRADED SAND: light yellowish brown; dry non-cohesive; dense; trace of gravel.		116.0	1.6
			BOTTOM			

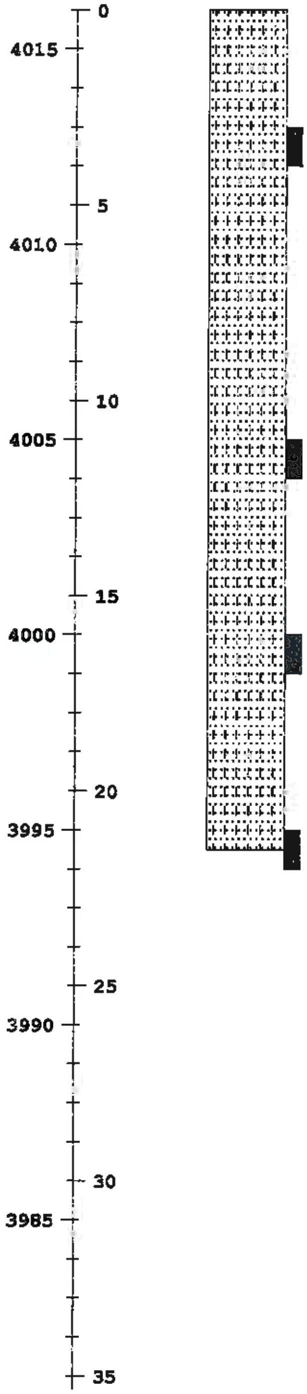
Figure Number 2

LOG OF TEST BORING
BORING B-2

Page 1 of 1

PROJECT: *PINON HILLS DOLLAR GENERAL STORE*
BORING DATE: *6/25/2105*
BORING LOCATION: *SEE BORING LOCATION MAP, FIGURE 1*
DRILL METHOD: *4-1/4 INCH I.D. HOLLOW-STEM AUGER*
DESCRIPTION: *GEOTECHNICAL INVESTIGATION*
DEPTH TO WATER - ⊻ **:** *N/A*
CAVING - ➧ **:** *N/A*

FILE NO: *15-15400*
ELEV.: *4016'*
START: *6/25/2015*
FINISH: *6/25/2015*
LOGGER: *MIKE WATTS*



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Remarks	Density pcf	Moisture %
		SW-SM	WELL-GRADED SAND with low fine content: light yellowish brown; dry; fine to medium grained; rock; non-cohesive.			
	7/6 9/6 14/6		medium dense; rock		118.1	0.8
	10/6 18/6 30/6		dense		123.9	0.9
	6/6 17/6 33/6		very dense		123.9	1.1
	12/6 14/6 12/6		medium dense		117.9	2.4
	12/6 20/6 32/6		very dense		116.0	2.4
			BOTTOM			

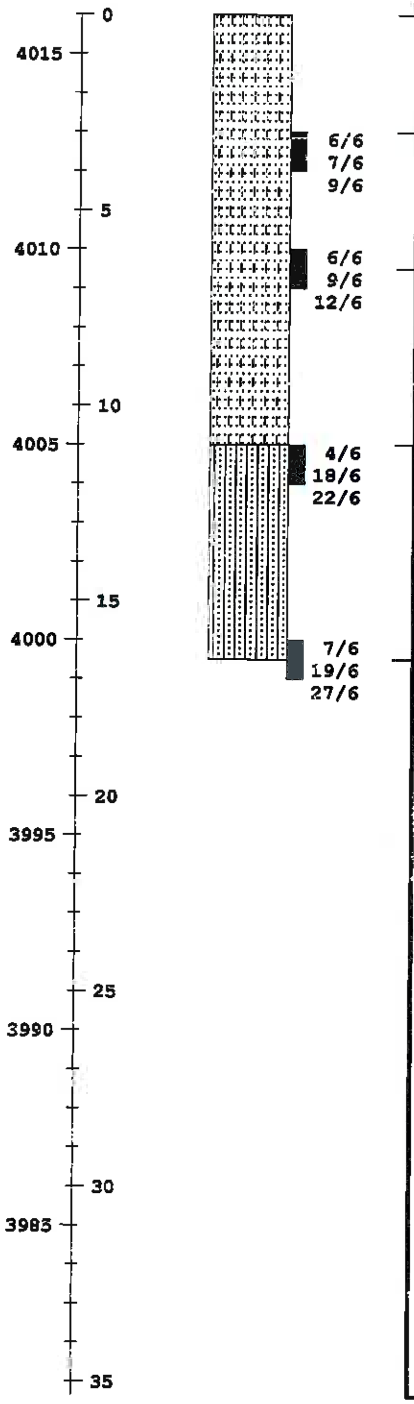
Figure Number 3

LOG OF TEST BORING
BORING B-3

Page 1 of 1

PROJECT: *PINON HILLS DOLLAR GENERAL STORE*
BORING DATE: *6/25/2105*
BORING LOCATION: *SEE BORING LOCATION MAP, FIGURE 1*
DRILL METHOD: *4-1/4 INCH I.D. HOLLOW-STEM AUGER*
DESCRIPTION: *GEOTECHNICAL INVESTIGATION*
DEPTH TO WATER - ▽ **:** *N/A*
CAVING - ► **:** *N/A*

FILE NO: *15-15400*
ELEV.: *4016'*
START: *6/25/2015*
FINISH: *6/25/2015*
LOGGER: *MIKE WATTS*



SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Remarks	Density pcf	Moisture %
	SW-SM	WELL-GRADED SAND with low fine content: light yellowish brown; dry; fine to medium grained; rock.			
6/6 7/6 9/6		medium dense; non-cohesive		115.1	1.2
6/6 9/6 12/6		slow drilling due to rock		117.2	1.2
4/6 18/6 22/6	SM	SILTY SAND: yellowish brown; cohesive; dry; dense; trace gravel		128.8	1.5
7/6 19/6 27/6		BOTTOM		118.1	1.9

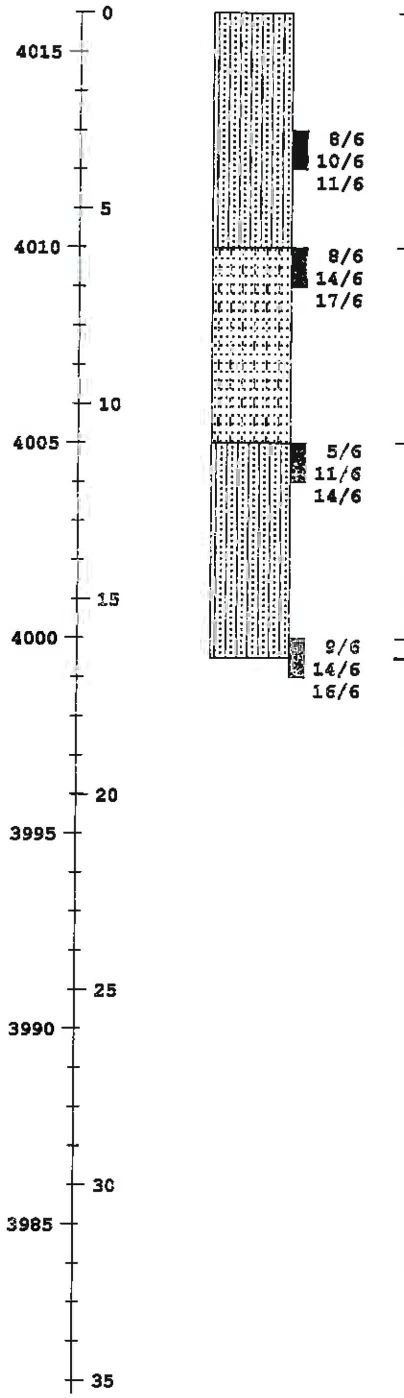
Figure Number 4

LOG OF TEST BORING
BORING B-4

Page 1 of 1

PROJECT: *PINON HILLS DOLLAR GENERAL STORE*
BORING DATE: *6/25/2105*
BORING LOCATION: *SEE BORING LOCATION MAP, FIGURE 1*
DRILL METHOD: *4-1/4 INCH I.D. HOLLOW-STEM AUGER*
DESCRIPTION: *GEOTECHNICAL INVESTIGATION*
DEPTH TO WATER - ▼ **:** *N/A*
CAVING - ► **:** *N/A*

FILE NO: *15-15400*
ELEV.: *4016'*
START: *6/25/2015*
FINISH: *6/25/2015*
LOGGER: *MIKE WATTS*



SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Remarks	Density pcf	Moisture %
8/6 10/6 11/6	SM	SILTY SAND: light yellowish brown; dry; fine to medium grained; medium dense; slow drilling due to rock.		117.7	1.0
8/6 14/6 17/6	SW-SM	WELL GRADED SAND with low fine content: light yellowish brown; dry; dense; rock		131.6	1.6
5/6 11/6 14/6	SM	SILTY SAND: yellowish brown; dry; low cohesion; medium dense; fine.		118.9	1.6
9/6 14/6 16/6		dense		122.6	4.4
		BOTTOM			

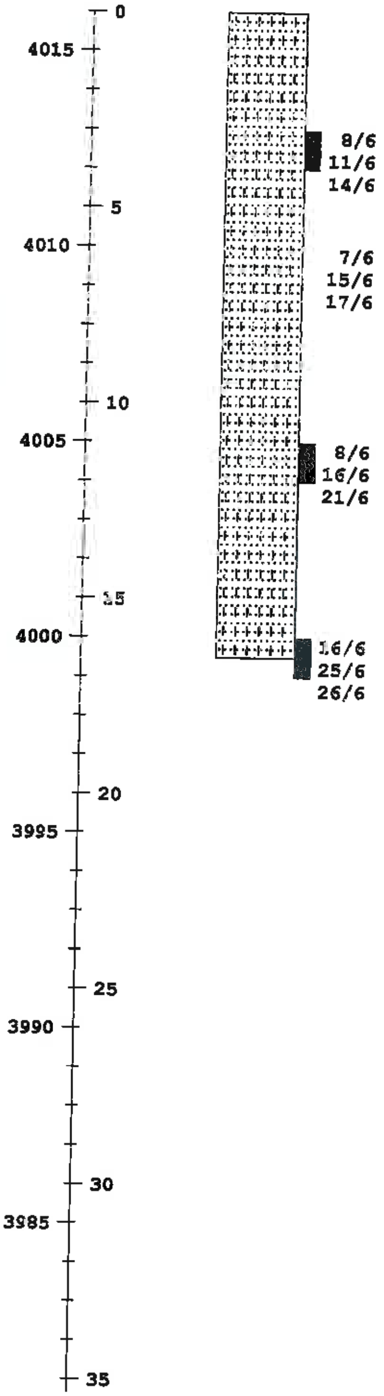
Figure Number 5

LOG OF TEST BORING
BORING B-5

Page 1 of 1

PROJECT: *PINON HILLS DOLLAR GENERAL STORE*
BORING DATE: *6/25/2105*
BORING LOCATION: *SEE BORING LOCATION MAP, FIGURE 1*
DRILL METHOD: *4-1/4 INCH I.D. HOLLOW-STEM AUGER*
DESCRIPTION: *GEOTECHNICAL INVESTIGATION*
DEPTH TO WATER - ≖ : *N/A*
CAVING - ➧ : *N/A*

FILE NO: *15-15400*
ELEV.: *4016'*
START: *6/25/2015*
FINISH: *6/25/2015*
LOGGER: *MIKE WATTS*



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Remarks	Density pcf	Moisture %
	8/6 11/6 14/6	SW-SM	WELL-GRADED SAND with low fine content: light yellowish brown; dry; medium dense; non-cohesive; rock.		114.3	0.9
	7/6 15/6 17/6		No recovery due to rock	moisture can from sampler		1.1
	8/6 16/6 21/6		dense		122.8	1.1
	16/6 25/6 26/6		very dense BOTTOM		113.7	2.3

Figure Number 6

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
▨	Poorly graded sand
▨	Silty sand
▨	Well graded sand with silt
Soil Samplers	
■	California sampler

Notes:

1. Five Exploratory borings were drilled on 6/25/2105 using an 8in outside diameter hollow-stem auger.

2. No free water was encountered at the time of drilling to the maximum depth drilled of 21.5 feet.

3. Boring locations are shown on the Boring Location Map, Figure 1.

4. These logs are subject to the limitations, conclusions and recommendations in this report.

5. Results of tests conducted on samples recovered are reported on the logs

APPENDIX C

SOIL TEST DATA

SIEVE ANALYSES

Grain size distributions for samples selected as most representative of sub-soils encountered in our test borings were determined by sieve analysis (ASTM Test Method D422). Test results are shown in Figures A-1 through A-11.

IN-SITU MOISTURE RELATIONSHIPS (ASTM D 2216)

Moisture density data for disturbed native soils was obtained by use of a 2-3/8 inch (inside diameter) split-barrel sampler. Test results are given on the Test Boring Logs.

CONSOLIDATION TESTS (ASTM D 2435)

Compressibility of soils was determined on saturated, undisturbed samples of native materials. Consolidation Test Diagrams, Figure B-1, graphically expresses the relationship of vertical strain vs. applied vertical (normal) load for earth materials selected as most representative of the soil strata within the anticipated zone of influence of foundation loads.

DIRECT SHEAR TESTS (ASTM D 3080)

One (1) quick-consolidated direct shear test was performed on an undisturbed, saturated sample of native earth materials. This test provides information on soil shear strength vs. normal load and is used to determine the angle of internal friction and cohesion of earth materials under essentially drained conditions. Test results are presented in Figure C-1.

R-VALUE TESTS

Four (4) R-Value tests were performed in accordance with Test Method No. California 301-F to obtain flexible pavement design data. Test results are given in Figures D-1 thru D-4.

SOIL CORROSIVITY (SO_4 / pH / Chlorides)

Tests for Soluble Sulfates (SO_4), Soluble Chlorides (Cl), and pH values were performed on a composite sample taken from the upper 3 feet of Boring 1, 2 & 3 to determine the corrosion potential of the soils. Corrosion prevention measures and the extent to which measures should be taken (if any) should be addressed with the corrosion engineer. Soluble Sulfates and Soluble Chlorides values were determined according to EPA 300.0M. The pH values were determined according to EPA 9045C. Results of all three constituents are discussed in the report, **Section V**.

PERCOLATION TESTS

Two (2) percolation tests were performed in accordance with the U.S. Public Health Service Test Procedure (Manual of Septic Tank Practice, Part I). Tests were taken at three (3) feet below the existing ground surface. Results are shown in Table 1.

SIMONCRE VIA SOLERI II, LLC

Geotechnical Soils Investigation
Pinon Hills Dollar General Store
~0.20 Miles NW of Oasis Road on the South Side of HWY 138, Pinon Hills, San Bernardino County, CA

SEI File No. 15-15400
July 13, 2015

TEST LOCATION	USCS	% < # 200	CONSOLIDATION				DIRECT SHEAR		UNCONFINED COMPRESSION		E.I.	ATTERBERG LIMITS			R-VALUE @ 300 psi		MAXIMUM DENSITY	
			C_C	C_s	S.P. (psf)	HV %	C, (ksf)	F.A.	Q_U, (psi)	C, (ksf)		LL	PL	PI	R.V.	E.P. (psi)	MDD (pcf)	O.M.
B-1 @ 3'	SM	13	0.04	0	0	-0.1												
B-2 @ 6'	SW-SM	8.2					0.34	39.8										
B-3 @ 3'	SW-SM	7.6																
B-4 @ 3'	SM	15																
B-4 @ 6'	SW-SM	5.9																
B-5 @ 3'	SW-SM	6.7																
B-5 @ 6'	SW-SM	9.3																
R-1 @ 0-5'	SP-SM	8.6													71	0		
R-2 @ 0-5'	SP-SM	9.3													67	0		
R-3 @ 0-5'	SM	14													67	0		
R-4 @ 0-5'	SM	12													64	0		

CONSOLIDATION
Cc - Compression Index
Cs - Swell Index
S.P. (psf) - Swell Pressure
HV % - Heave Precentage / Collapase

UNCONFINED COMPRESSION
Q_U (psi) - Unconfined Compression Strength
C, (ksf) - Cohesion

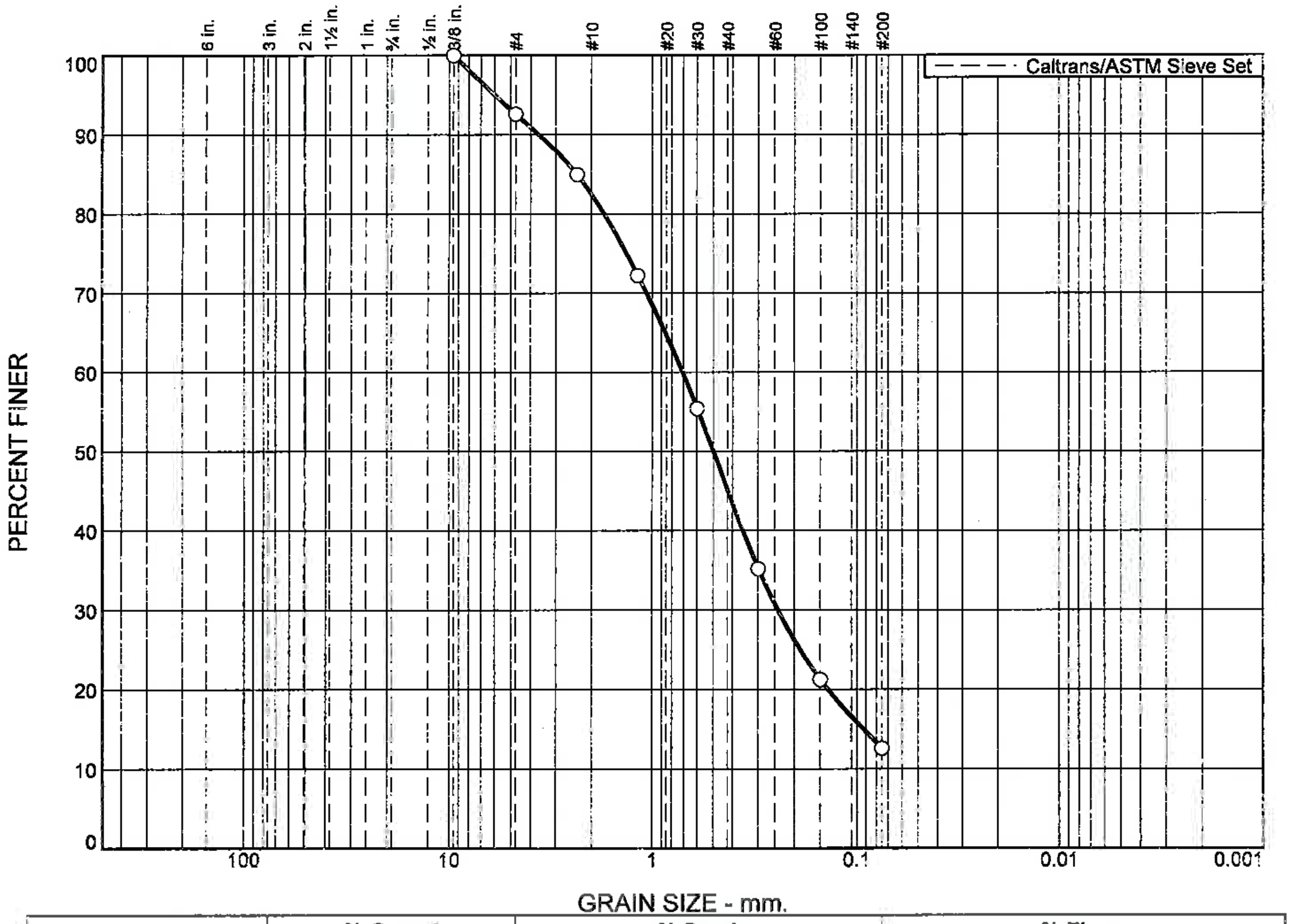
DIRECT SHEAR
C (ksf) - Cohesion
F.A. - Friction Angle

E.I. - EXPANSION INDEX
ATTERBERG LIMITS
LL - Liquid Limit
PL - Plastic Limit
PI - Plasticity Index

(R)ESISTANCE VALUE
RV - R-Value @ 300 psi
EV - Expansion Press @ 300 psi

MAXIMUM DENSITY
MDD (pcf) - Max Dry Density
O.M. - Optimum Moisture

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	7	11	37	32	13	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-1		3	SILTY SAND	SM

SOILS ENGINEERING, INC.

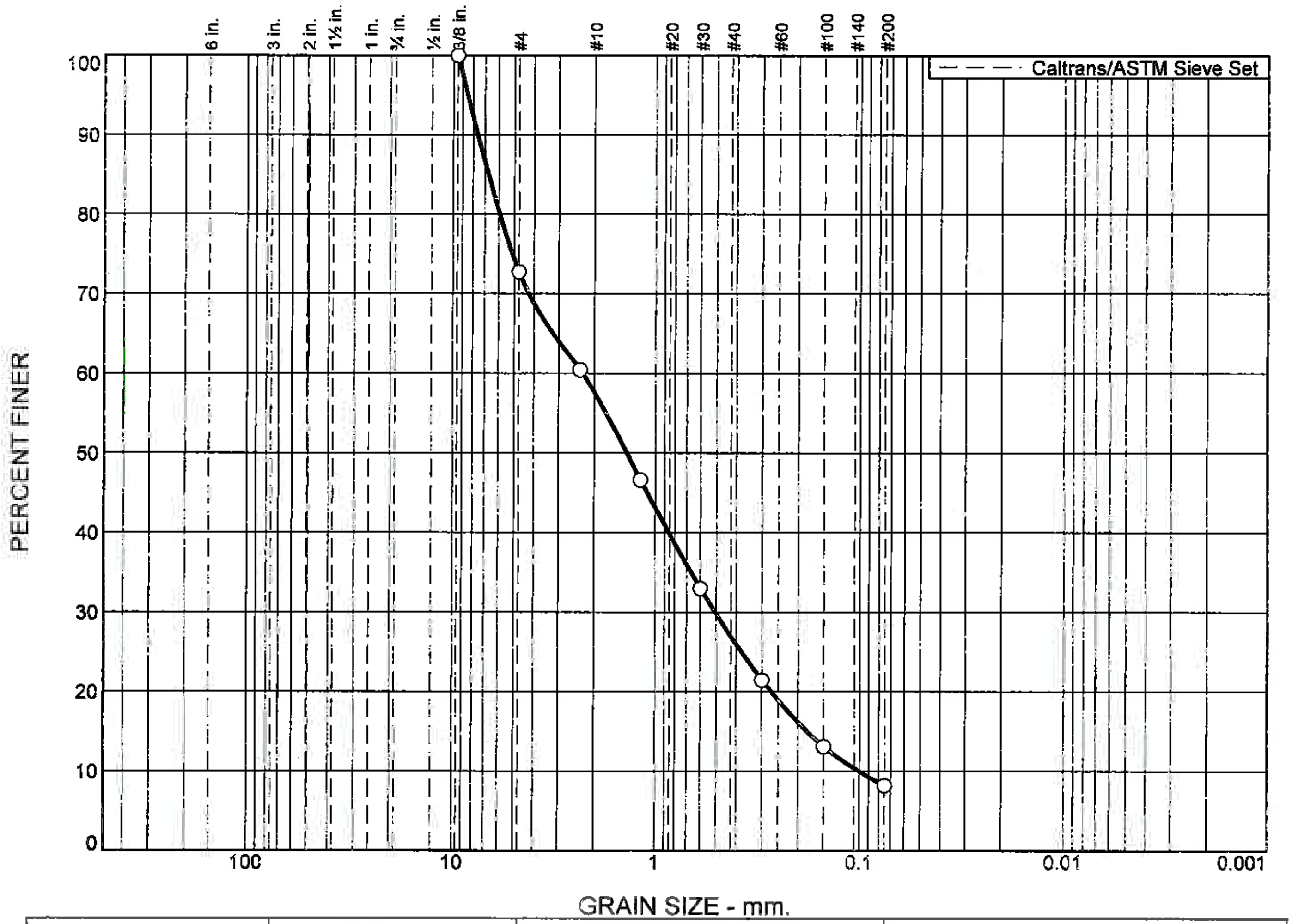
Client: SIMONCRE, SOLERI II, LLC

Project: PINON HILLS DOLLAR GENERAL STORE

Project No.: 15-15400

Figure A-1

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	27.3	15.1	30.7	18.7	8.2	

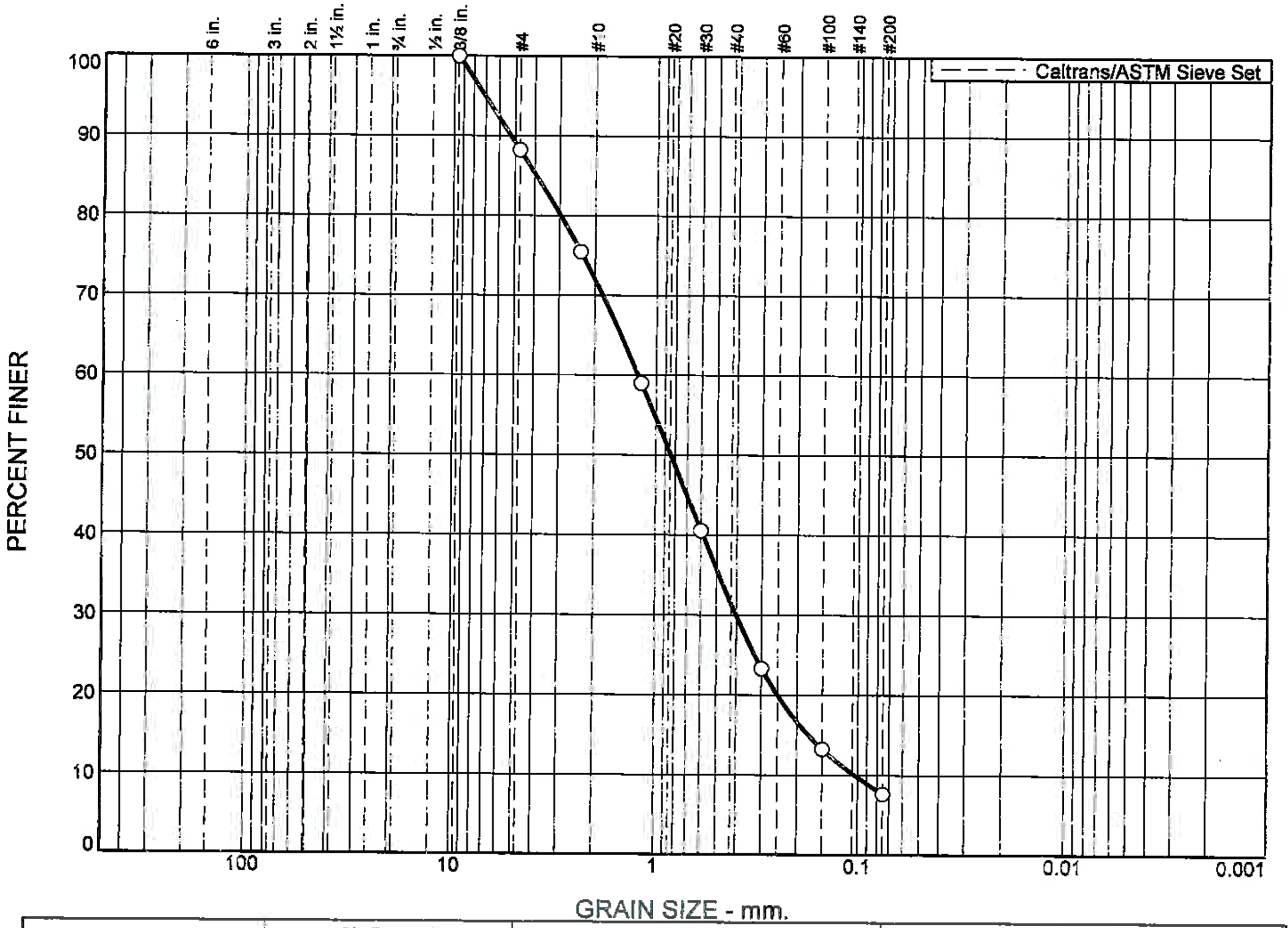
SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-2		6	WELL GRADED SAND with low fine content	SW-SM

SOILS ENGINEERING, INC.

Client: SIMONCRE, SOLERI II, LLC
Project: PINON HILLS DOLLAR GENERAL STORE
Project No.: 15-15400 **Figure** A-2

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	11.8	16.3	40.5	23.8	7.6	

SOIL DATA

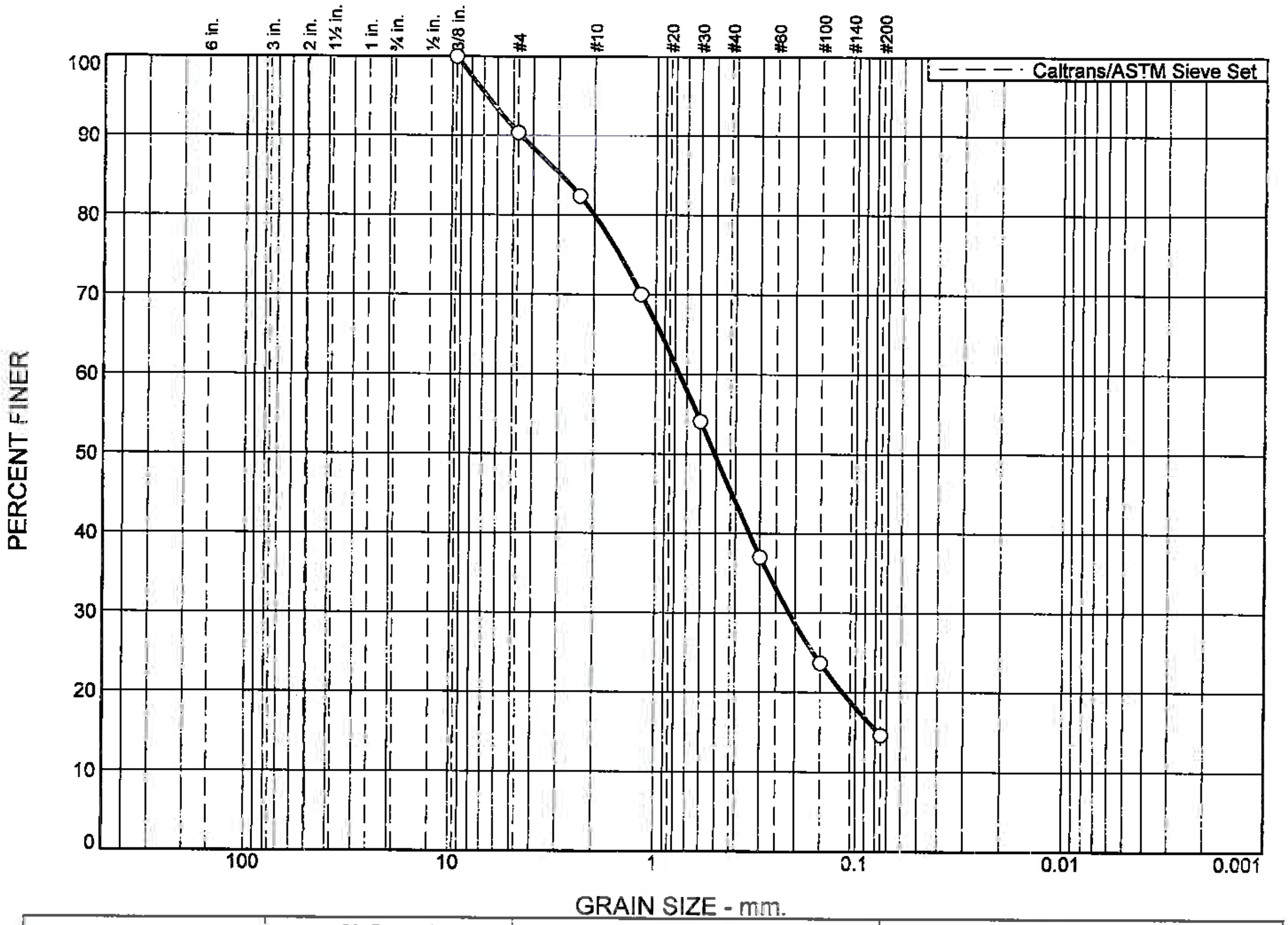
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-3		3	WELL GRADED SAND with low fine content	SW-SM

SOILS ENGINEERING, INC.

Client: SIMONCRE, SOLERI II, LLC
Project: PINON HILLS DOLLAR GENERAL STORE

Project No.: 15-15400 **Figure** A-3

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	10	10	35	30	15	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-4		3	SILTY SAND	SM

SOILS ENGINEERING, INC.

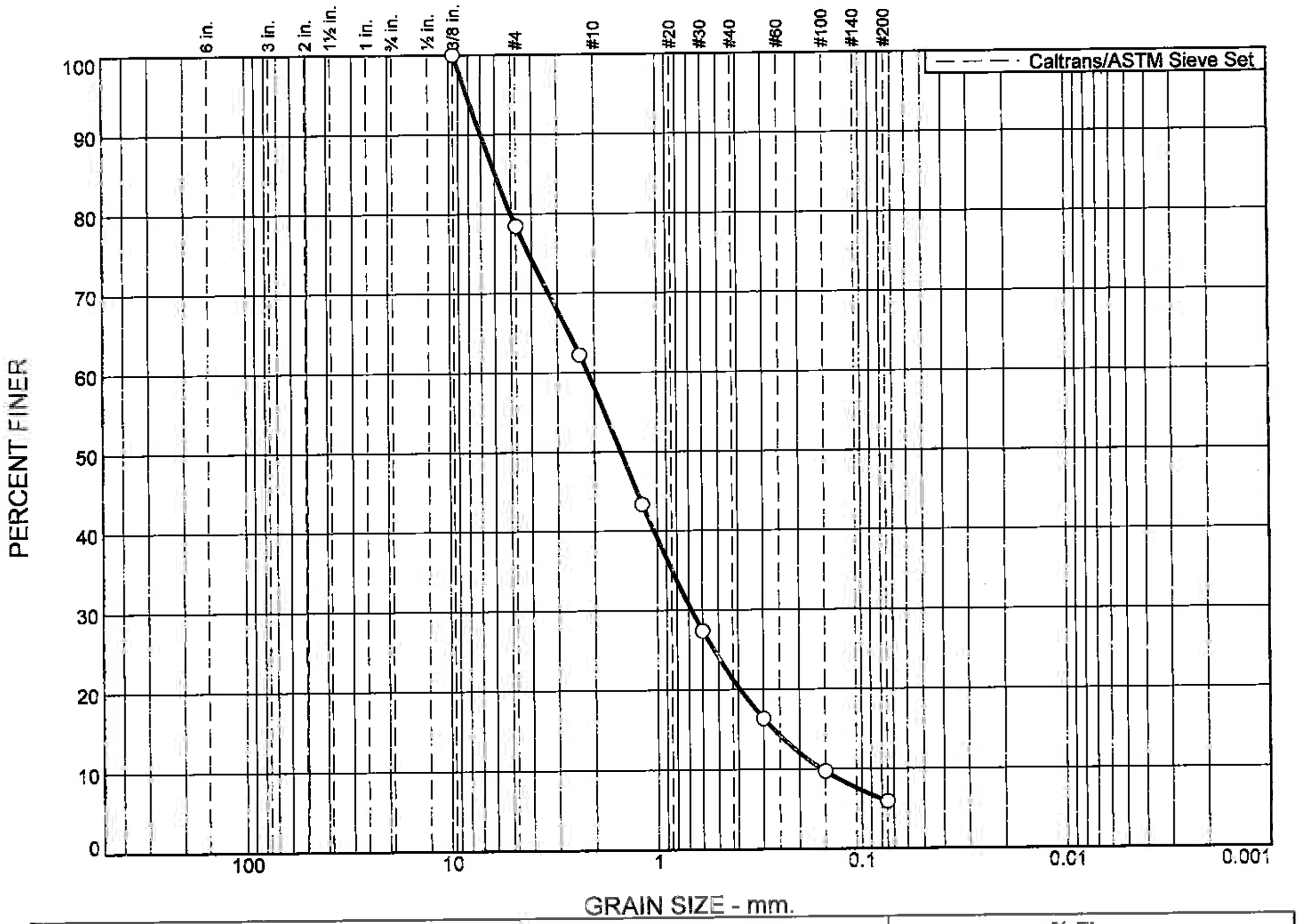
Client: SIMONCRE, SOLERI II, LLC

Project: PINON HILLS DOLLAR GENERAL STORE

Project No.: 15-15400

Figure A-4

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	21.5	20.5	36.7	15.4	5.9	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-4		6	WELL GRADED SAND with low fine content	SW-SM

SOILS ENGINEERING, INC.

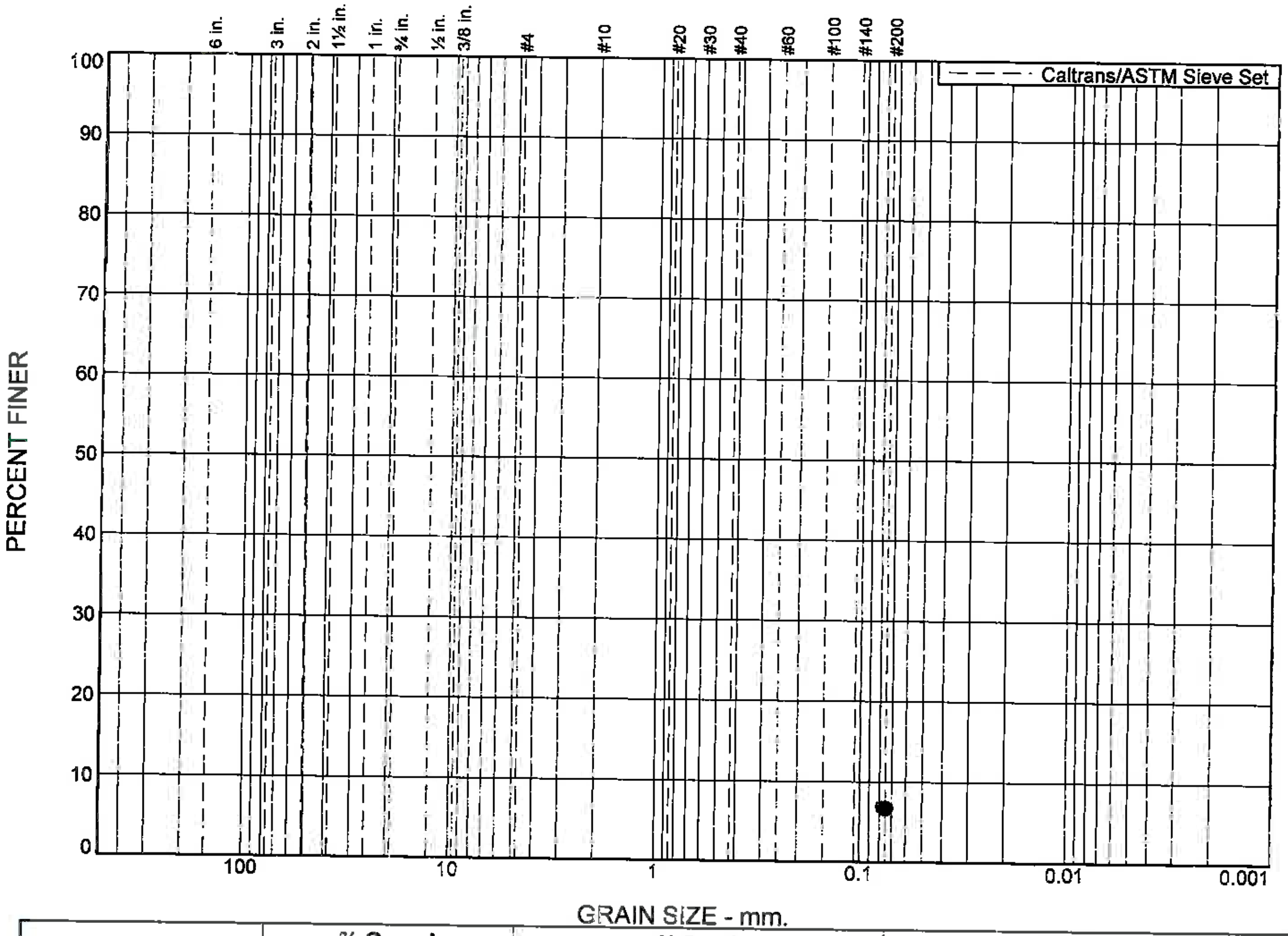
Client: SIMONCRE, SOLERI II, LLC

Project: PINON HILLS DOLLAR GENERAL STORE

Project No.: 15-15400

Figure A-5

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○							6.7	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-5		3	WELL GRADED SAND with low fine content	SW-SM

SOILS ENGINEERING, INC.

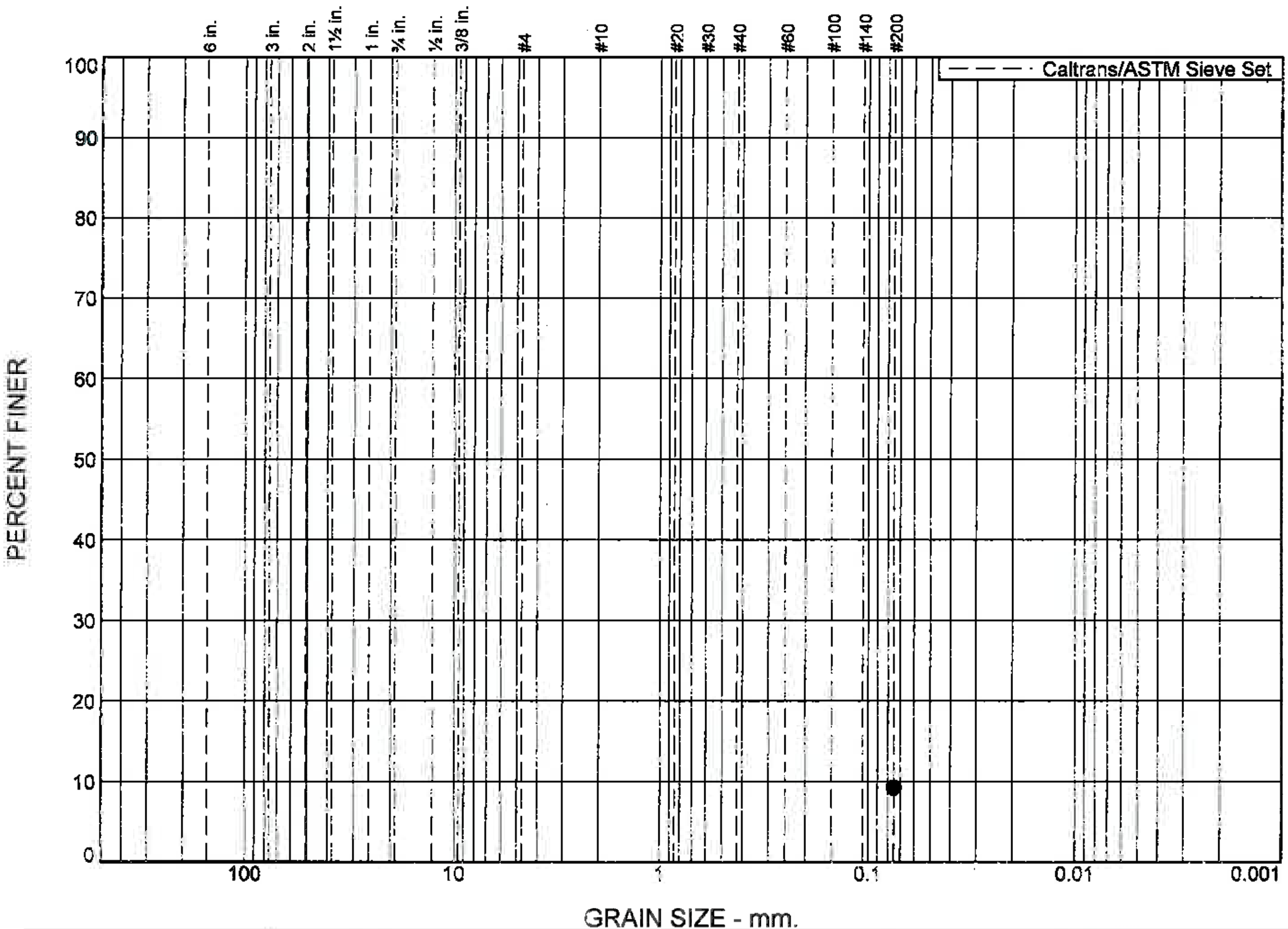
Client: SIMONCRE, SOLERI II, LLC

Project: PINON HILLS DOLLAR GENERAL STORE

Project No.: 15-15400

Figure A-6

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○							9.3	

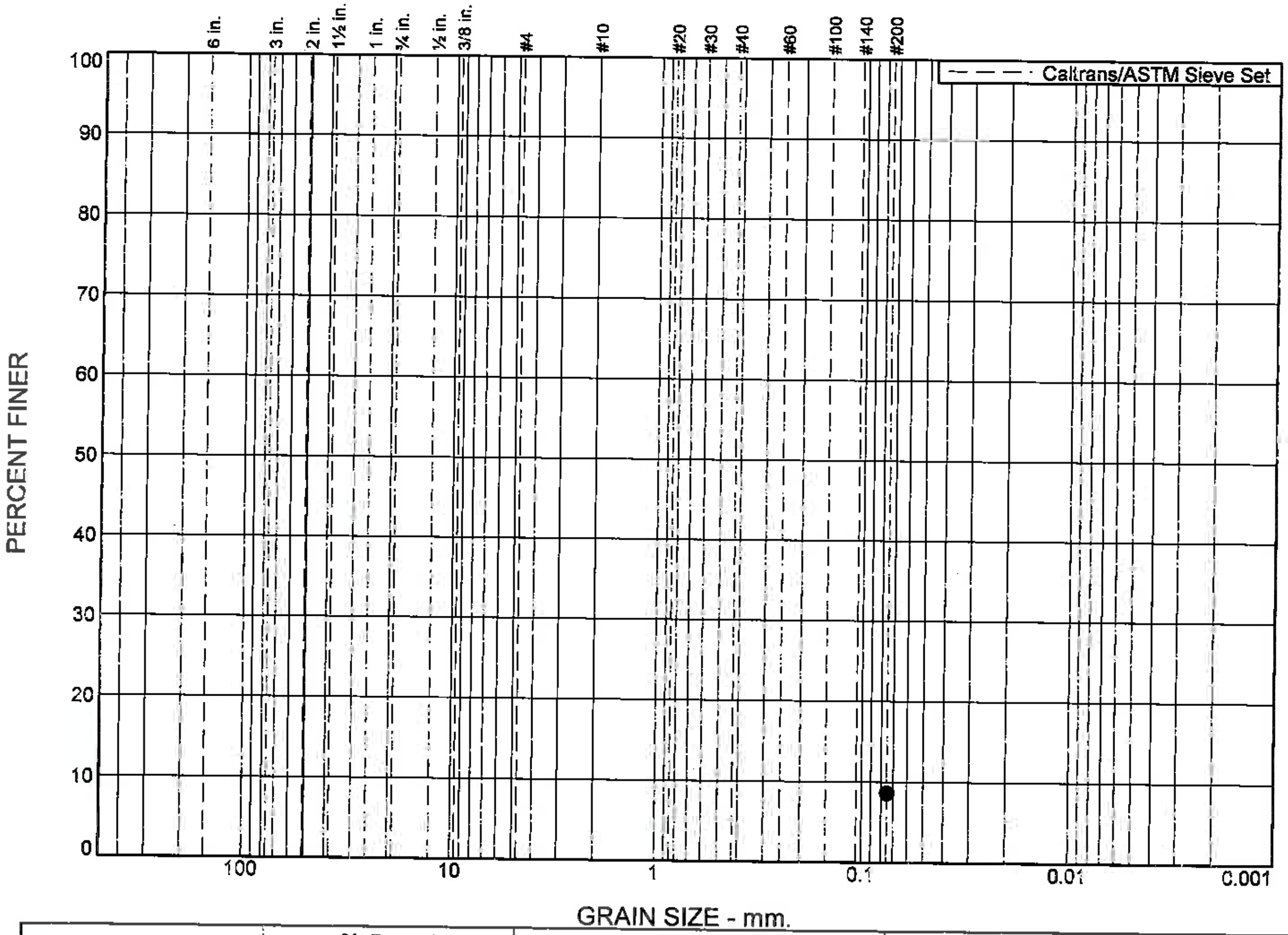
SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-5		6	WELL GRADED SAND with low fine content	SW

SOILS ENGINEERING, INC.

Client: SIMONCRE, SOLERI II, LLC
Project: PINON HILLS DOLLAR GENERAL STORE
Project No.: 15-15400 **Figure** A-7

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○							8.6	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○		52151	0-5'	POORLY GRADED SAND with low fine content	SP-SM
				Location: R-1 @ 0-5'	
				Large Bulk Sample	

SOILS ENGINEERING, INC.

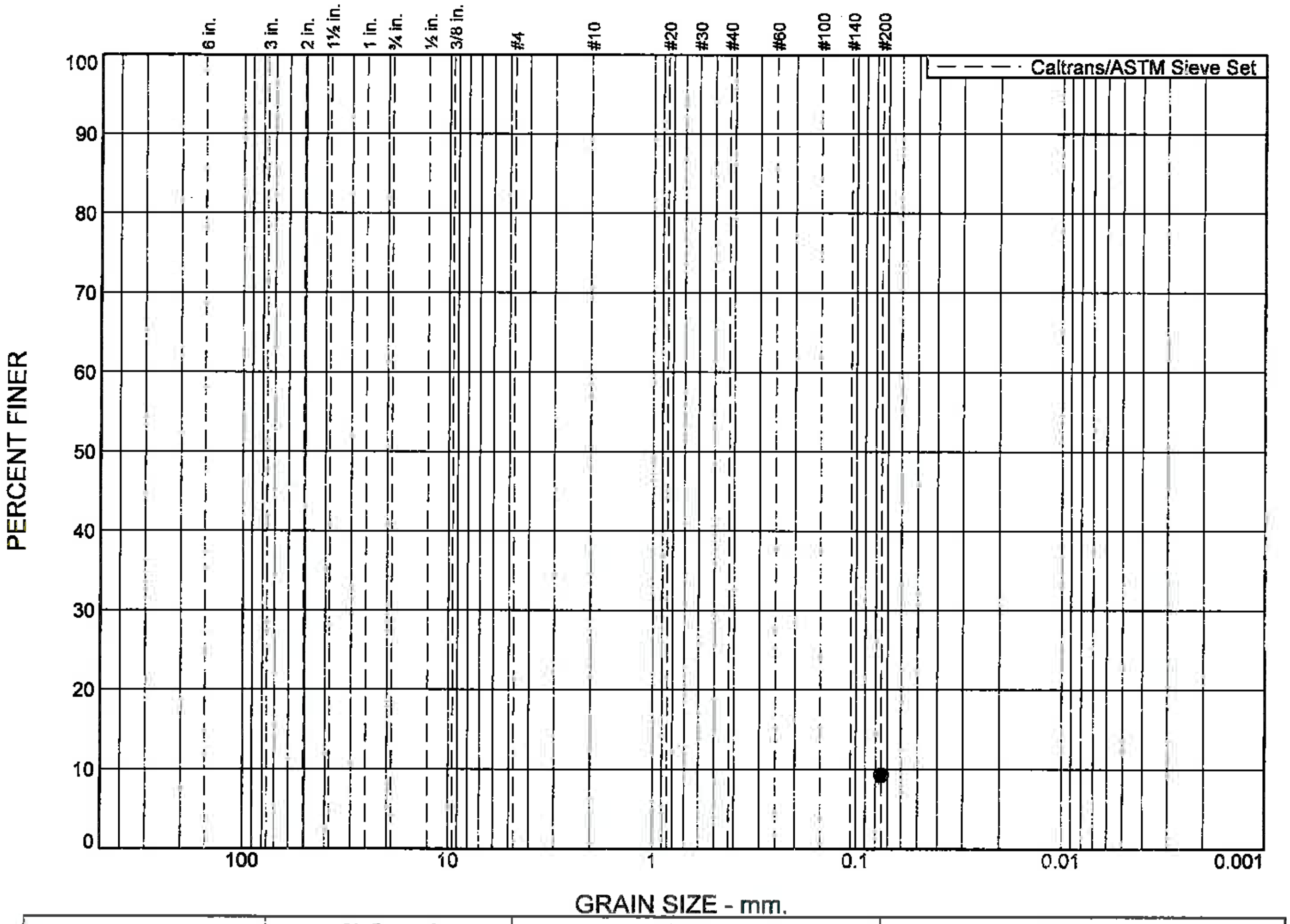
Client: SIMONCRE, SOLERI II, LLC

Project: PINON HILLS DOLLAR GENERAL STORE

Project No.: 15-15400

Figure A-8

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○							9.3	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○		52153	0-5'	POORLY GRADED SAND with low fine content	SP-SM
				Location: R-2 @ 0-5'	
				Large Bulk Sample	

SOILS ENGINEERING, INC.

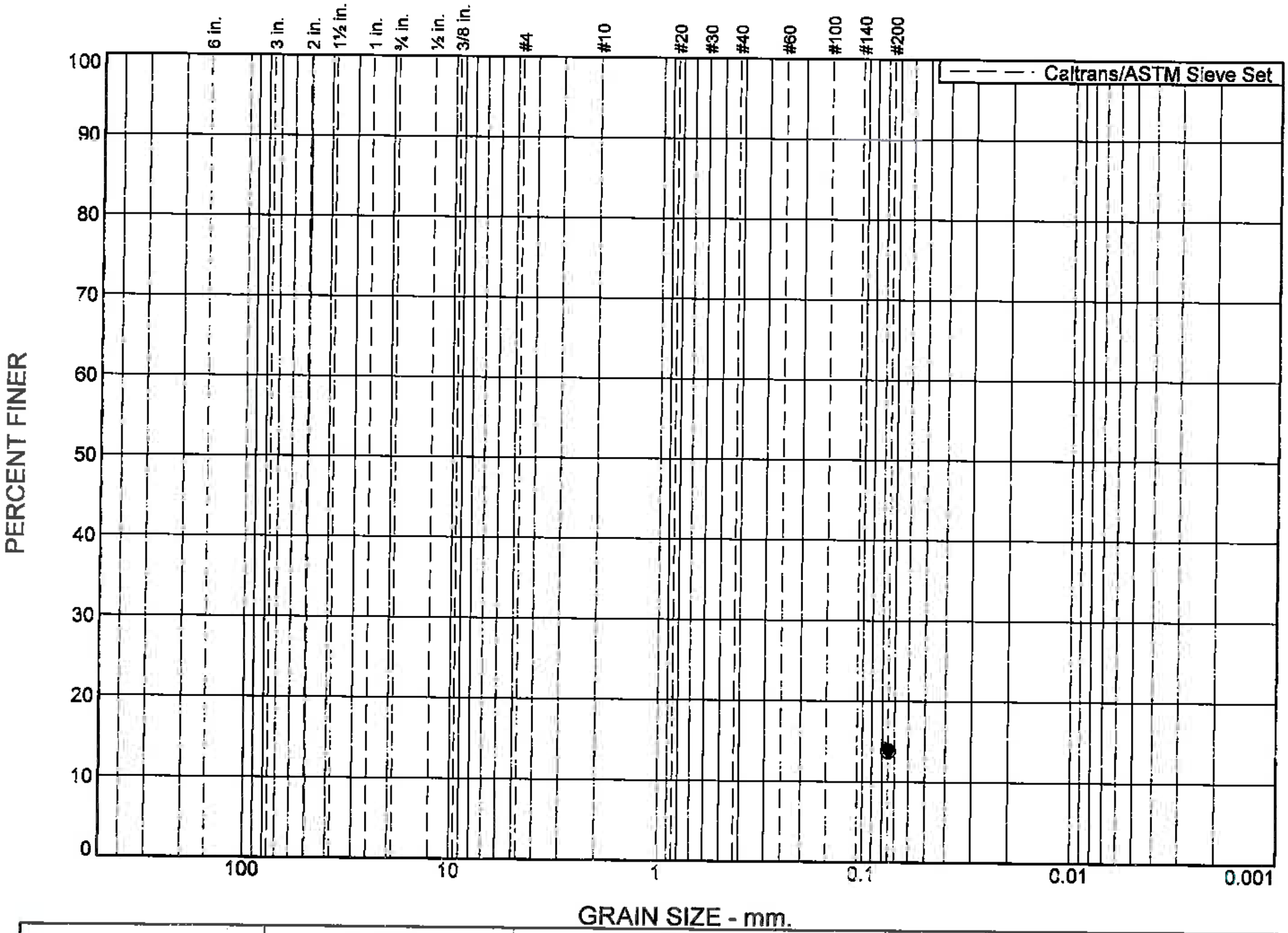
Client: SIMONCRE, SOLERI II, LLC

Project: PINON HILLS DOLLAR GENERAL STORE

Project No.: 15-15400

Figure A-9

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○							14	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○		52154	0-5'	SILTY SAND	SM
				Location: R-3 @ 0-5'	
				Large Bulk Sample	

SOILS ENGINEERING, INC.

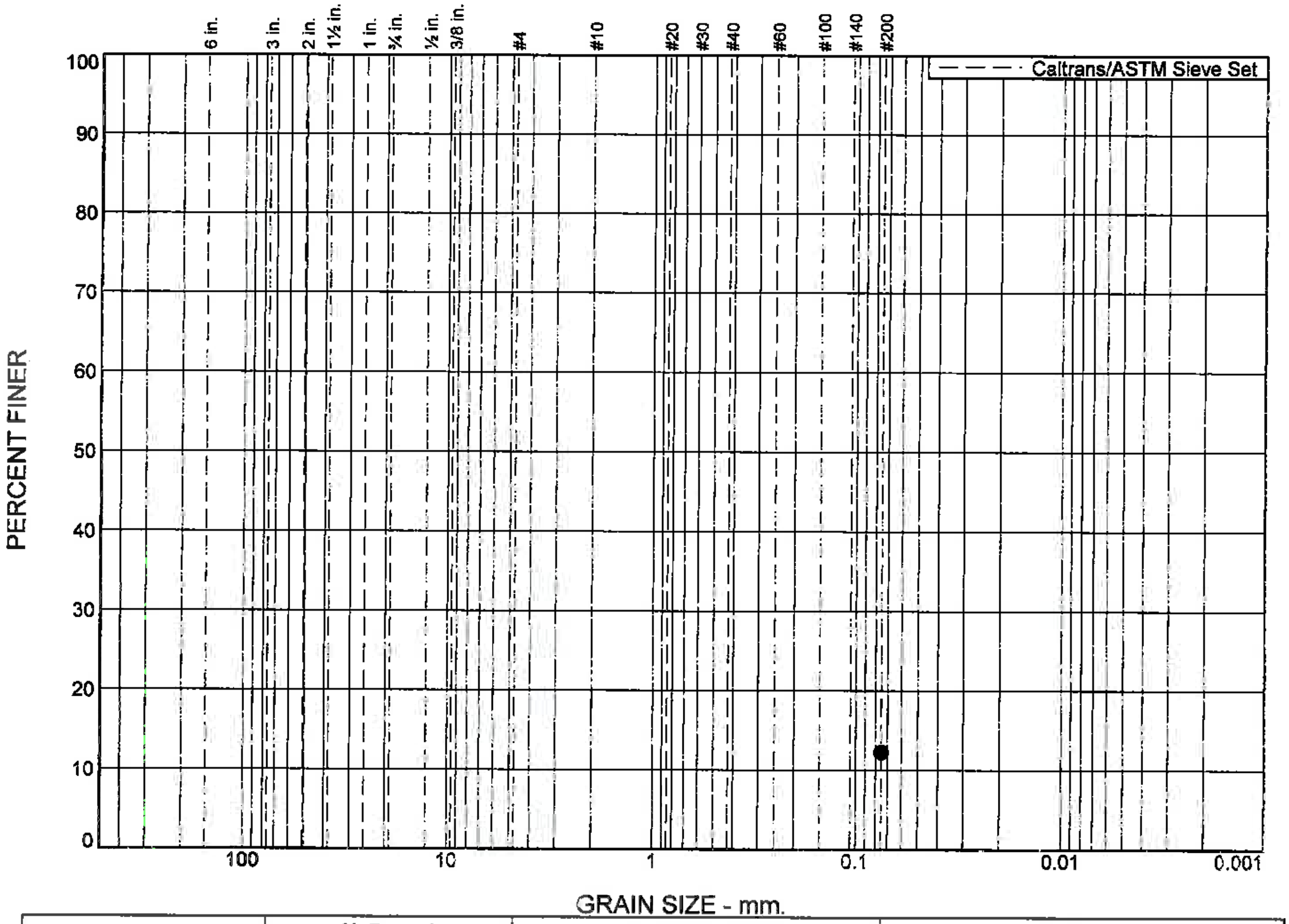
Client: SIMONCRE, SOLERI II, LLC

Project: PINON HILLS DOLLAR GENERAL STORE

Project No.: 15-15400

Figure A-10

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○							12	

SOIL DATA

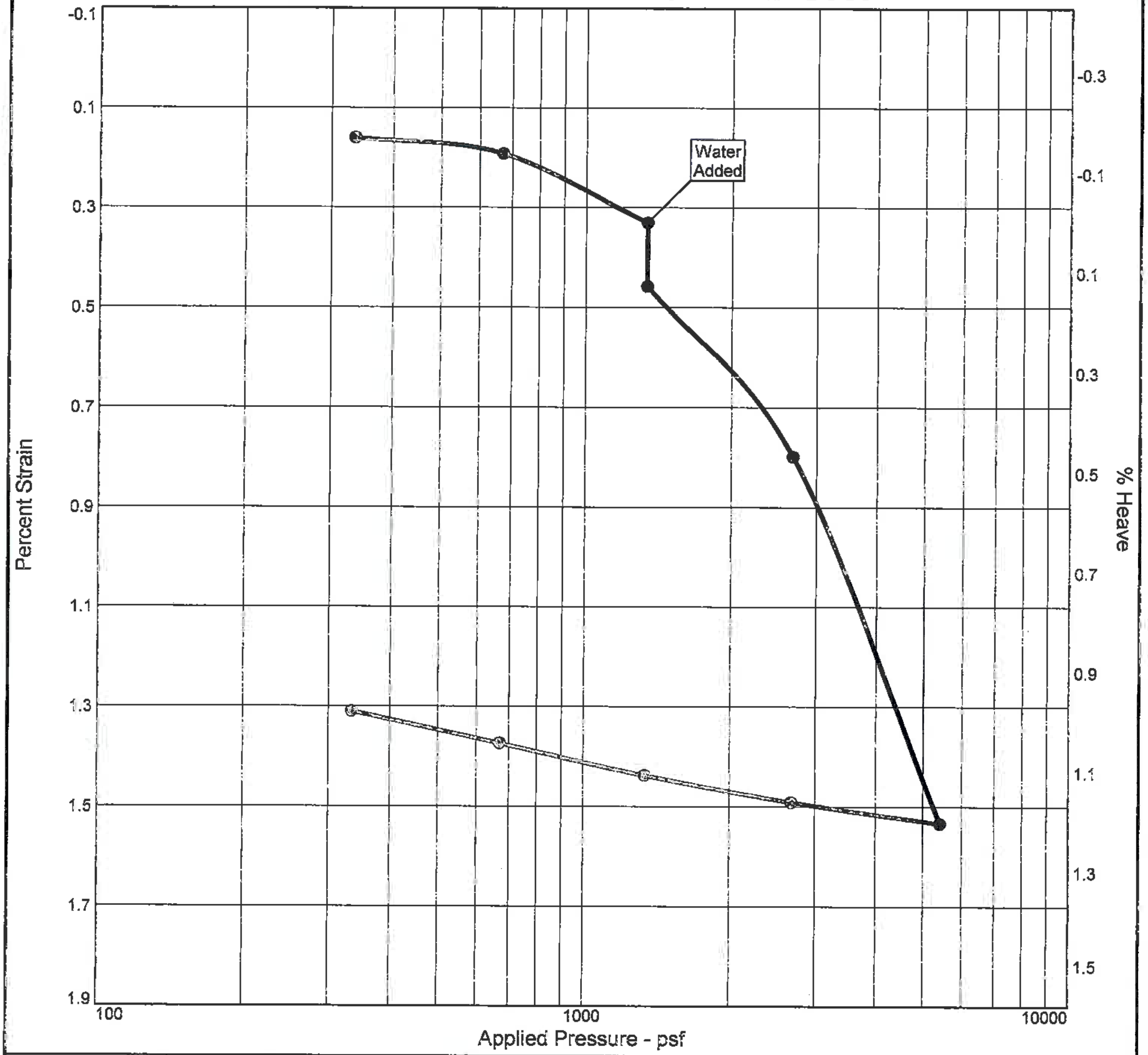
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○		52155	0-5'	SILTY SAND	SM
				Location: R-4 @ 0-5'	
				Large Bulk Sample	

SOILS ENGINEERING, INC.

Client: SIMONCRE, SOLERI II, LLC
Project: PINON HILLS DOLLAR GENERAL STORE

Project No.: 15-15400 **Figure** A-11

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_s	Swell Press. (psf)	Heave %	e_o
Sat.	Moist.											
29.0 %	6.5 %	103.6	N/A	N/A	2.65	336	2615	0.04	0.00		-0.1	0.596

MATERIAL DESCRIPTION	USCS	AASHTO
SILTY SAND	SM	N/A

Project No. 15-15400 **Client:** SIMONCRE, SOLERI II, LLC

Project: PINON HILLS DOLLAR GENERAL STORE

Source of Sample: B-1 **Depth:** 3

Remarks:

Test Date: 07/10/15

Tested By: RG

Sample No: 52075

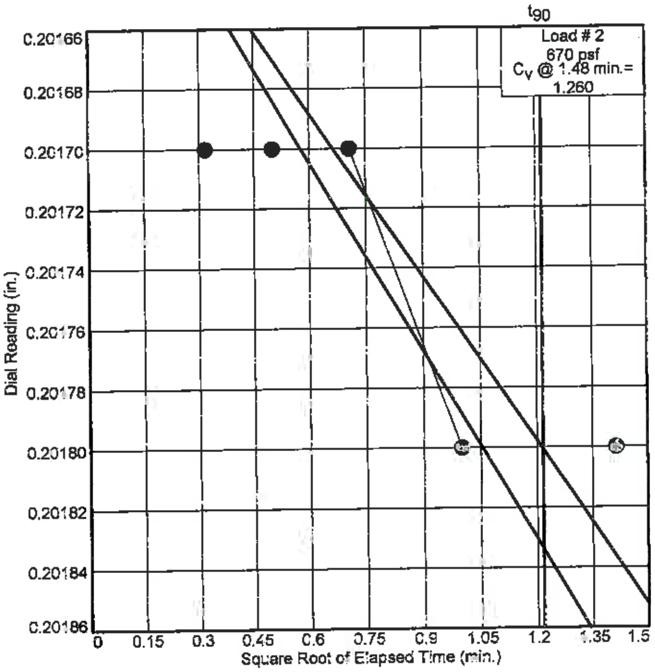
SOILS ENGINEERING, INC.

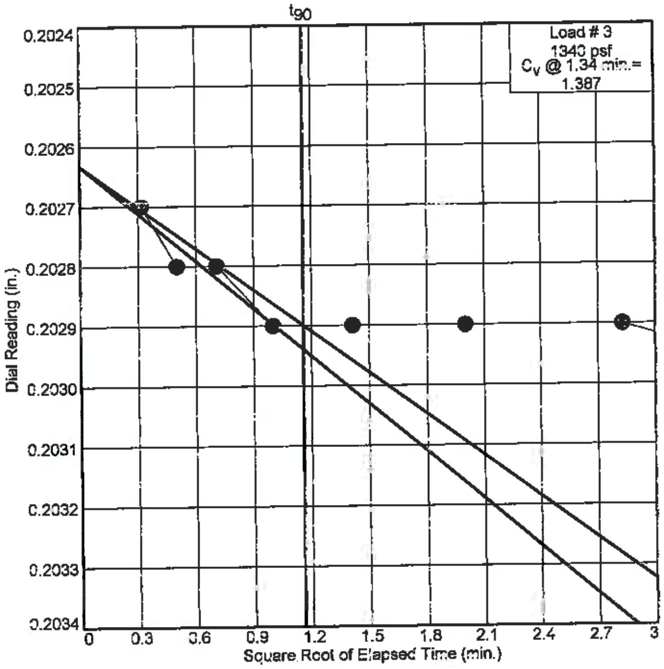
Figure B-1

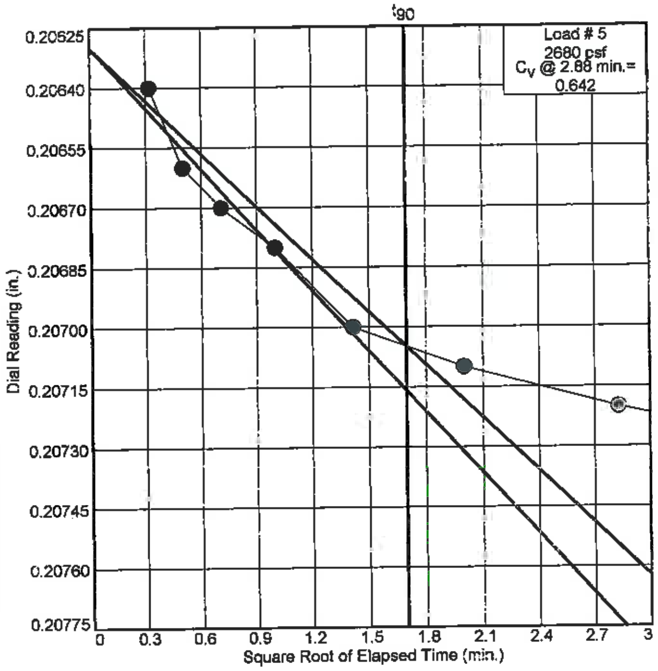
Dial Reading vs. Time

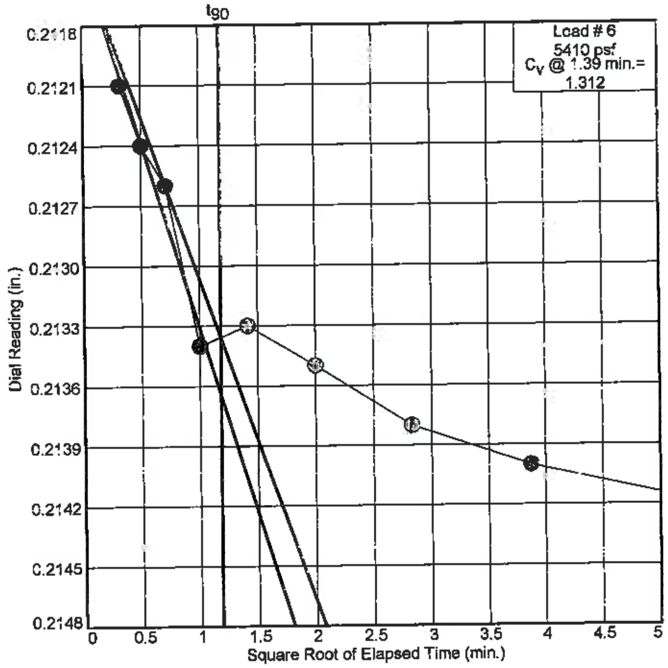
Project No.: 15-15400
Project: PINON HILLS DOLLAR GENERAL STORE

Source of Sample: B-1 Depth: 3



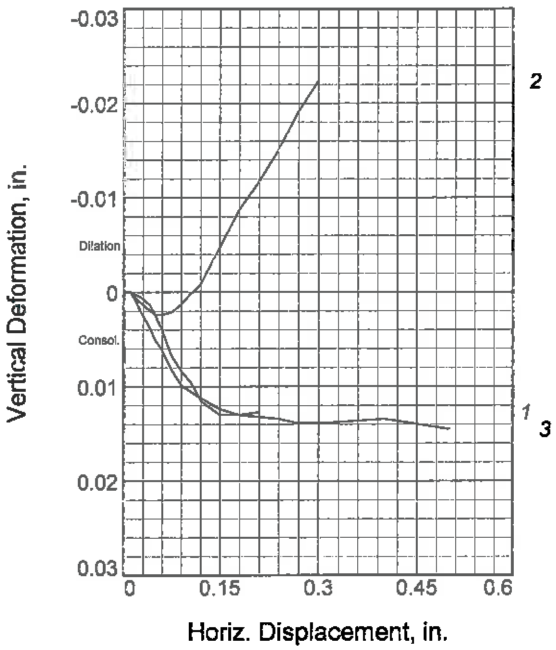


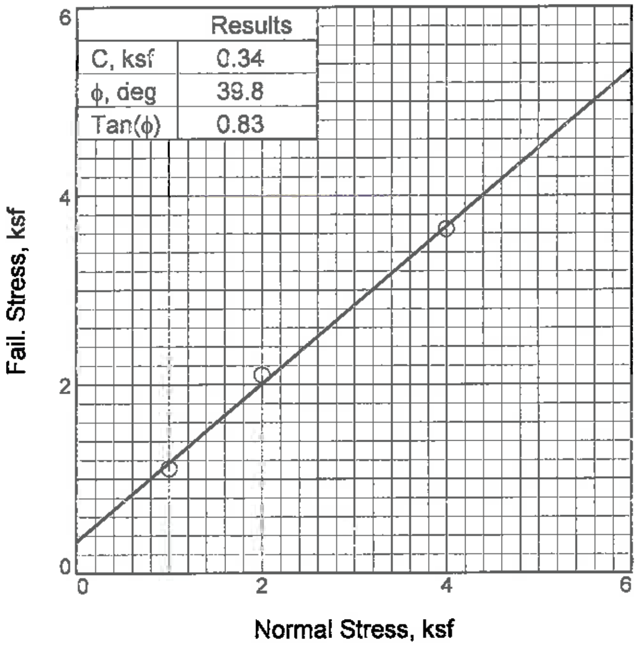




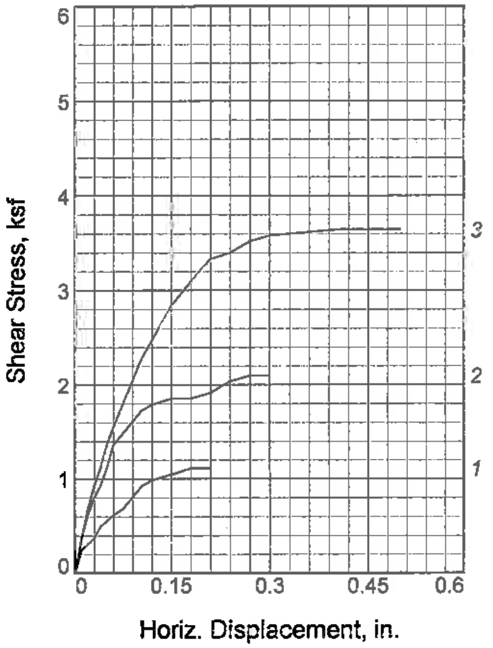
SOILS ENGINEERING, INC.

Figure B-1





	Results
C, ksf	0.34
ϕ, deg	39.8
Tan(ϕ)	0.83



Sample No.		1	2	3
Initial	Water Content, %	5.6	4.8	5.5
	Dry Density, pcf	107.5	118.8	110.4
	Saturation, %	27.5	32.3	29.0
	Void Ratio	0.5391	0.3921	0.4983
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	15.8	14.3	14.6
	Dry Density, pcf	107.5	118.8	110.4
	Saturation, %	77.5	96.4	77.9
	Void Ratio	0.5391	0.3921	0.4983
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
Normal Stress, ksf		1.00	2.00	4.00
Fail. Stress, ksf		1.11	2.10	3.64
Displacement, in.		0.18	0.27	0.40
Ult. Stress, ksf				
Displacement, in.				
Strain rate, in./min.		N/A	N/A	N/A

Sample Type: 2.5" x 6" TUBE

Description: WELL GRADED SAND with low fine content; brown; dense with gravel

LL= NP **PI=** NP

Assumed Specific Gravity= 2.65

Remarks: Test Date: 07/11/15

Figure C-1

Client: SIMONCRE, SOLERI II, LLC

Project: PINON HILLS DOLLAR GENERAL STORE

Source of Sample: B-2 **Depth:** 6

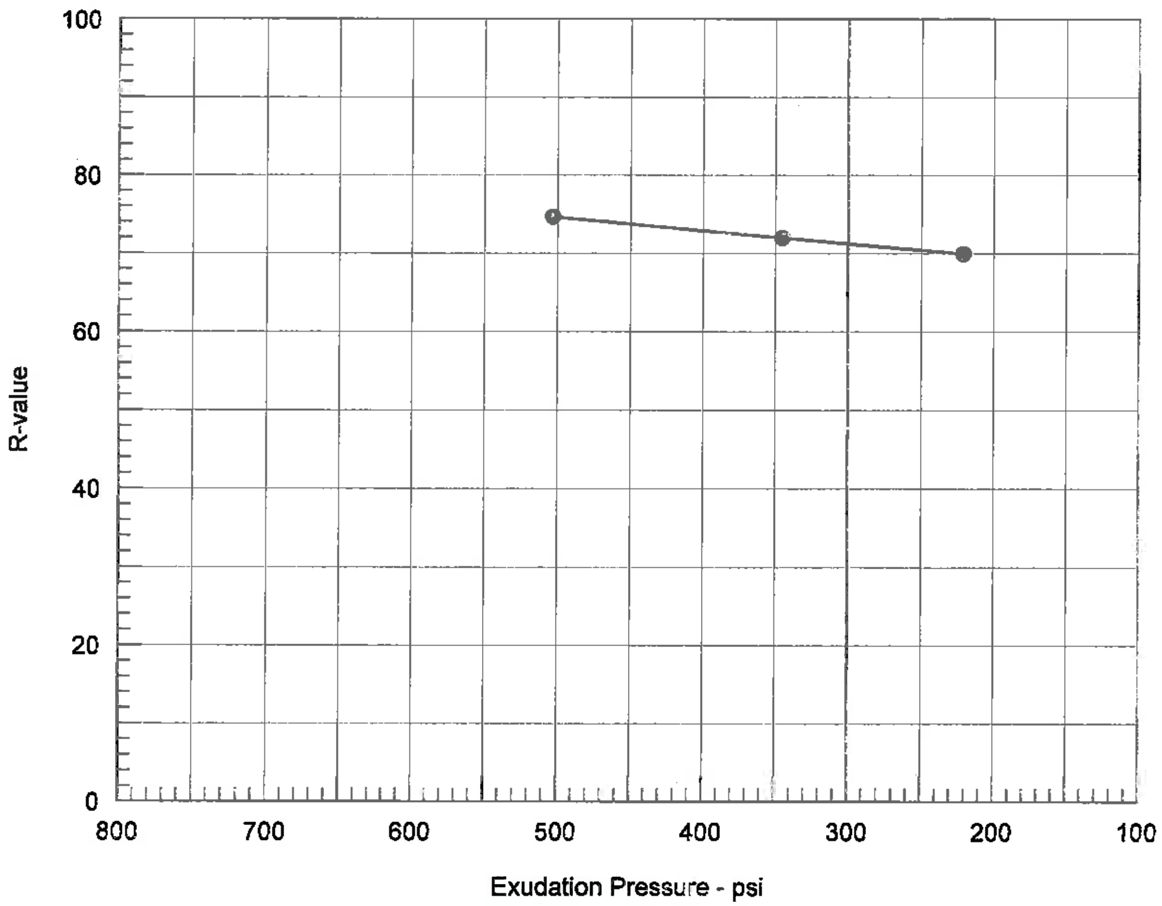
Proj. No.: 15-15400 **Date Sampled:** 06/25/15

DIRECT SHEAR TEST REPORT

SOILS ENGINEERING, INC.

Tested By: JMB **Checked By:** JW

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	125.9	7.0	0.00	21	2.53	503	75	75
2	350	126.4	7.5	0.00	23	2.51	345	72	72
3	350	124.9	8.5	0.00	26	2.48	221	70	70

Test Results	Material Description
R-value at 300 psi exudation pressure = 71	POORLY GRADED SAND with low fine content; Dark Brown, Trace of Gravel, Cohesive.

Project No.: 15-15400
Project: PINON HILLS DOLLAR GENERAL STORE
Location: R-1 @ 0-5'
Sample Number: 52151 **Depth:** 0-5'
Date: 7/13/2015

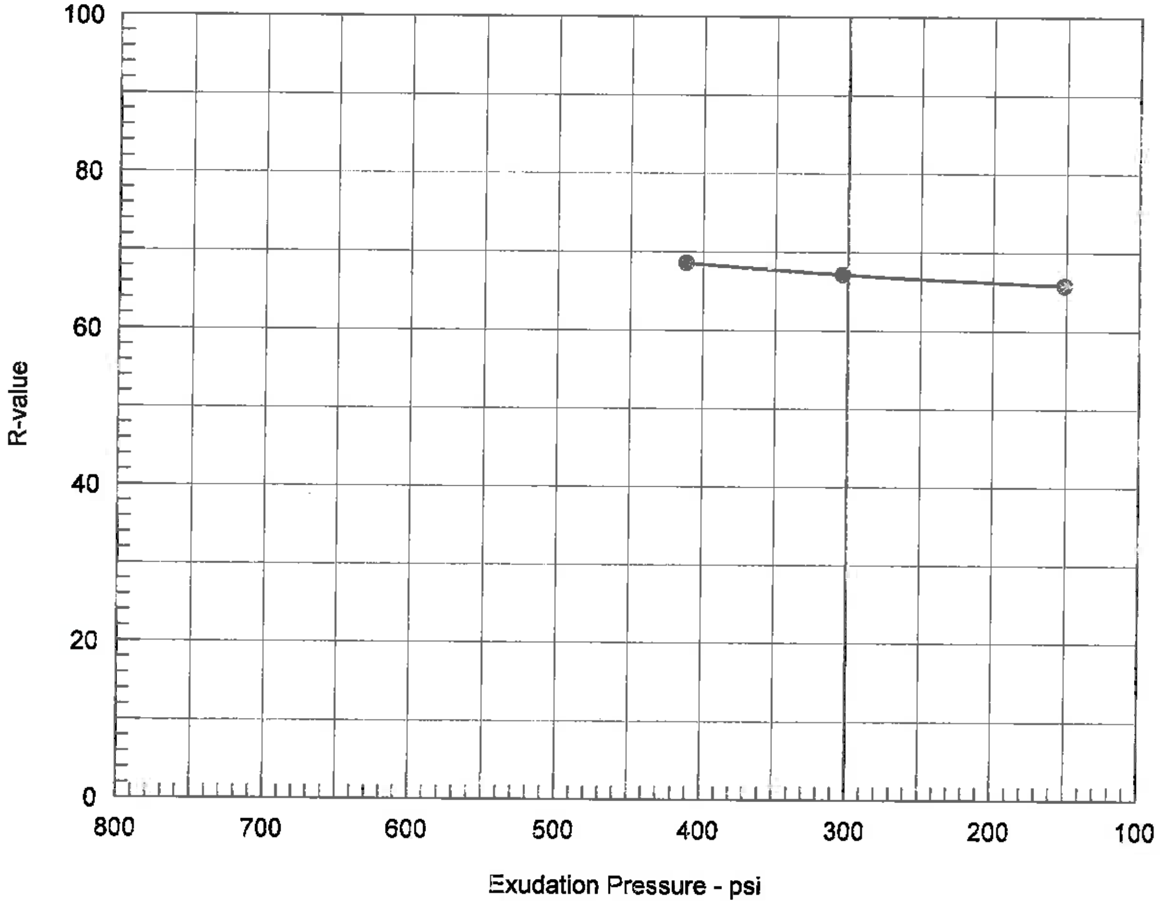
Tested by: JMB
Checked by: JW
Remarks:
Test Date: 07/08/15

R-VALUE TEST REPORT

SOILS ENGINEERING, INC.

Figure D-1

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	122.3	8.9	0.00	26	2.54	411	69	69
2	350	122.4	9.2	0.00	29	2.54	304	67	67
3	350	120.3	9.7	0.00	32	2.55	151	66	66

Test Results	Material Description
R-value at 300 psi exudation pressure = 67	POORLY GRADED SAND with low fine content; Dark Brown, Trace of Gravel, Cohesive.

Project No.: 15-15400
Project: PINON HILLS DOLLAR GENERAL STORE
Location: R-2 @ 0-5'
Sample Number: 52153 **Depth:** 0-5'
Date: 7/13/2015

Tested by: JMB
Checked by: JW

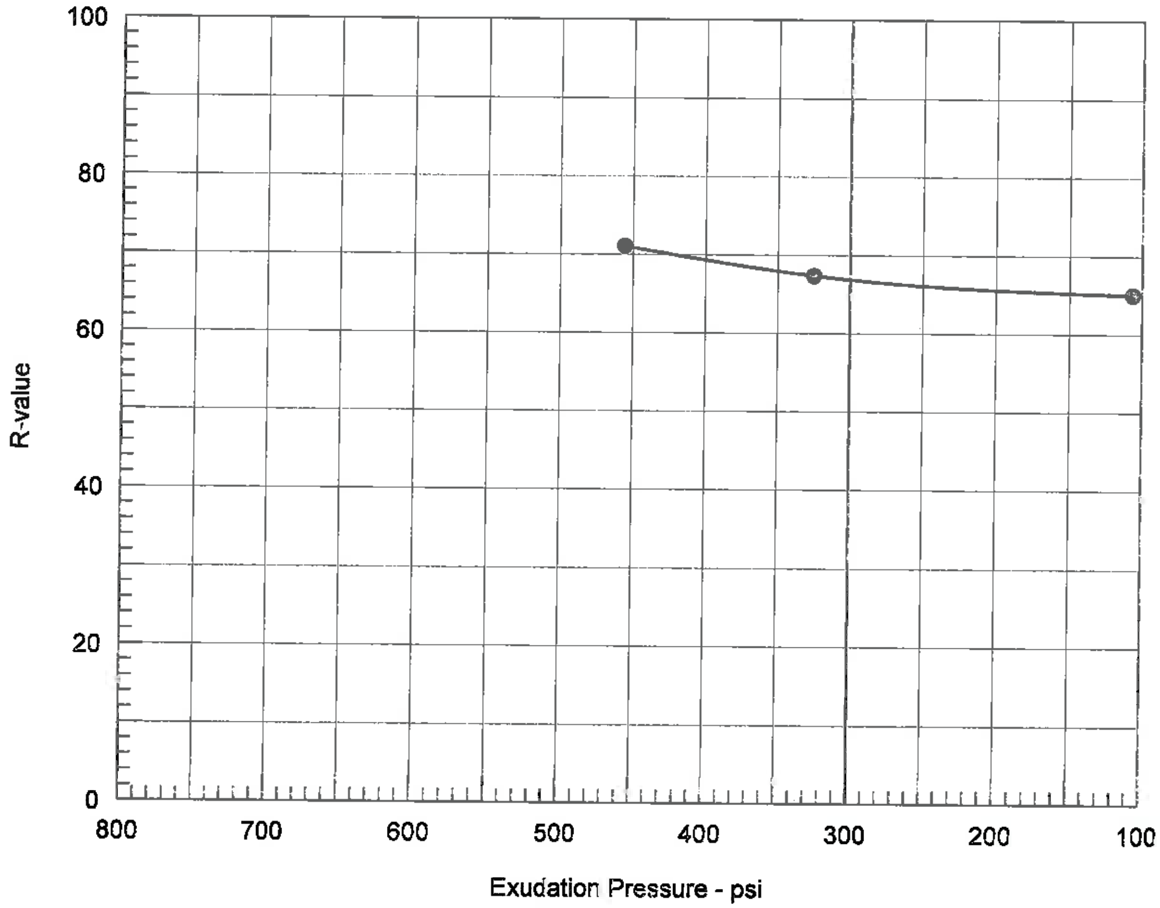
Remarks:
Test Date: 07/09/15

R-VALUE TEST REPORT

SOILS ENGINEERING, INC.

Figure D-2

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	124.2	8.4	0.00	26	2.53	456	71	71
2	350	122.1	8.9	0.00	28	2.53	325	67	67
3	350	120.3	9.9	0.00	30	2.53	106	65	65

Test Results	Material Description
R-value at 300 psi exudation pressure = 67	SILTY SAND; Dark Brown, Poorly Graded, Trace of Gravel, Cohesive.

Project No.: 15-15400

Project: PINON HILLS DOLLAR GENERAL STORE

Location: R-3 @ 0-5'

Sample Number: 52154 **Depth:** 0-5'

Date: 7/13/2015

Tested by: JMB

Checked by: JW

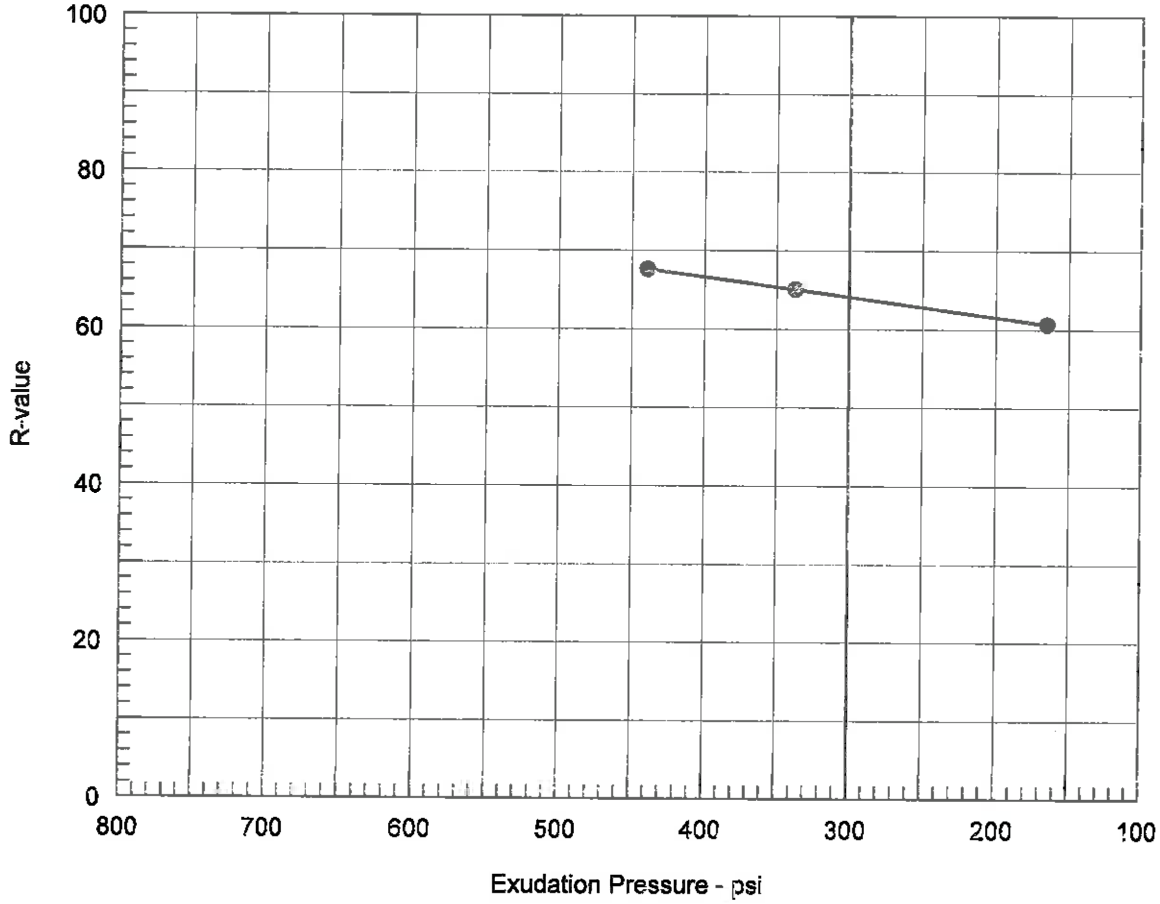
Remarks:
Test Date: 07/10/15

R-VALUE TEST REPORT

SOILS ENGINEERING, INC.

Figure D-3

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	118.8	9.9	0.00	30	2.58	439	66	68
2	350	119.9	10.4	0.00	33	2.56	337	64	65
3	350	117.7	10.9	0.00	38	2.57	165	59	61

Test Results	Material Description
R-value at 300 psi exudation pressure = 64	SILTY SAND; Dark Brown, Poorly Graded, Trace of Gravel, Cohesive.

Project No.: 15-15400

Project: PINON HILLS DOLLAR GENERAL STORE

Location: R-4 @ 0-5'

Sample Number: 52155 **Depth:** 0-5'

Date: 7/13/2015

Tested by: JMB

Checked by: JW

Remarks:
Test Date: 07/10/15

R-VALUE TEST REPORT

SOILS ENGINEERING, INC.

Figure D-4

TABLE 1

SOILS ENGINEERING, INC.

PERCOLATION TEST DATA LOG

SITE ADDRESS: Hwy 138 0.2 Miles NW of Oasis Road, Pinon Hills, CA

APN: 3068-231-38 & 3068-231-39 TEST PERFORMED BY: Soils Engineering, Inc. (SEI)

TEST DATE: 06/25/15 TEST HOLES WERE PRESATURATED FOR 16 HOURS

HOLE #	1				2				3			
DEPTH	5 FEET				5 FEET							
	TIME (MIN)		WATER LEVEL DROP (IN)	PERC RATE (MIN/IN)	TIME (MIN)		WATER LEVEL DROP (IN)	PERC RATE (MIN/IN)	TIME (MIN)		WATER LEVEL DROP (IN)	PERC RATE (MIN/IN)
	INITIAL	FINAL			INITIAL	FINAL			INITIAL	FINAL		
	TEST # 1				**TEST # 1**				**TEST # 1**			
	0	5	1.00		0	5	2.00					
	5	10	0.75		5	10	1.50					
	10	15	0.75		10	15	1.25					
	15	20	0.50		15	20	1.25					
	20	25	0.50		20	25	1.25					
	25	30	0.50	8	25	30	1.25	4				
	TEST # 2				**TEST # 2**				**TEST # 2**			
	0	5	0.50		0	5	1.25					
	5	10	0.50		5	10	1.00					
	10	15	0.50		10	15	1.00					
	15	20	0.50		15	20	1.00					
	20	25	0.25		20	25	1.00					
	25	30	0.25	12	25	30	1.00	5				

REMARKS: A total of two tests were ran for each location. Each test was ran for 30 min after presoak period.

MINIMUM OF 2 TEST HOLES REQUIRED. AVERAGE PERC RATE MAY BE USED IF 5 OR MORE TEST PER HOLE ARE PERFORMED OTHERWISE SLOWEST PERC RATE SHALL BE USED.

FINAL RATE TO BE USED IN DESIGN: 12 MINUTES PER INCH SOIL TYPE 1 2 3 [✓ 4] 5

LOUIS THOMAS BA
No. 000125

SIGNATURE OF QUALIFIED PROFESSIONAL:

APPENDIX D

SEISMIC INVESTIGATION

SEISMIC DESIGN INFORMATION
USGS Design Map Summary and Detail Report

EQFAULT
Version 3.00

California Fault Map

USGS Design Maps Summary Report

User–Specified Input

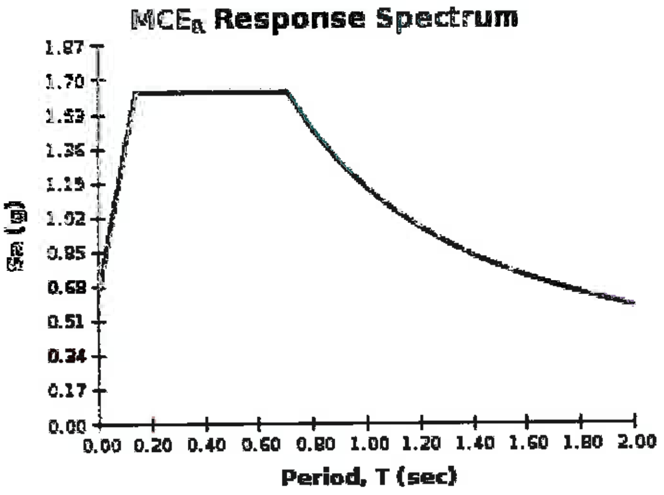
Report Title	15400 Dollar General Mon June 29, 2015 21:44:35 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.44128°N, 117.64536°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

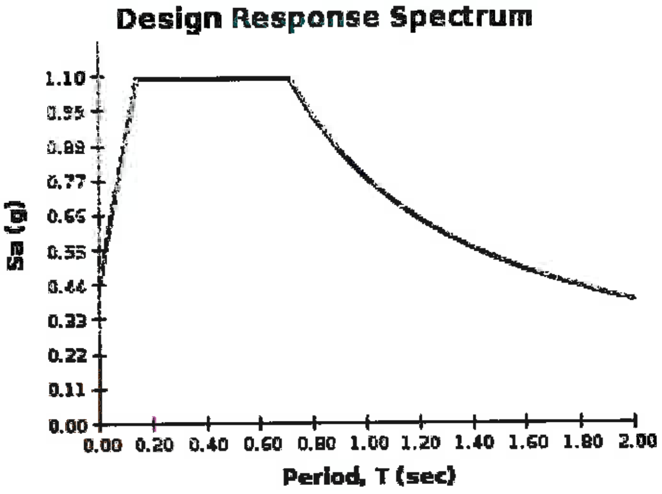
USGS–Provided Output

$S_S =$ 1.640 g $S_{MS} =$ 1.640 g $S_{DS} =$ 1.093 g

$S_1 =$ 0.775 g $S_{M1} =$ 1.163 g $S_{D1} =$ 0.775 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.44128°N, 117.64536°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.640$ g

From Figure 22-2 [2] $S_1 = 0.775$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.640 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.775 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.640 = 1.640$ g

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Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.775 = 1.163$ g

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Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.640 = 1.093$ g

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Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.163 = 0.775$ g

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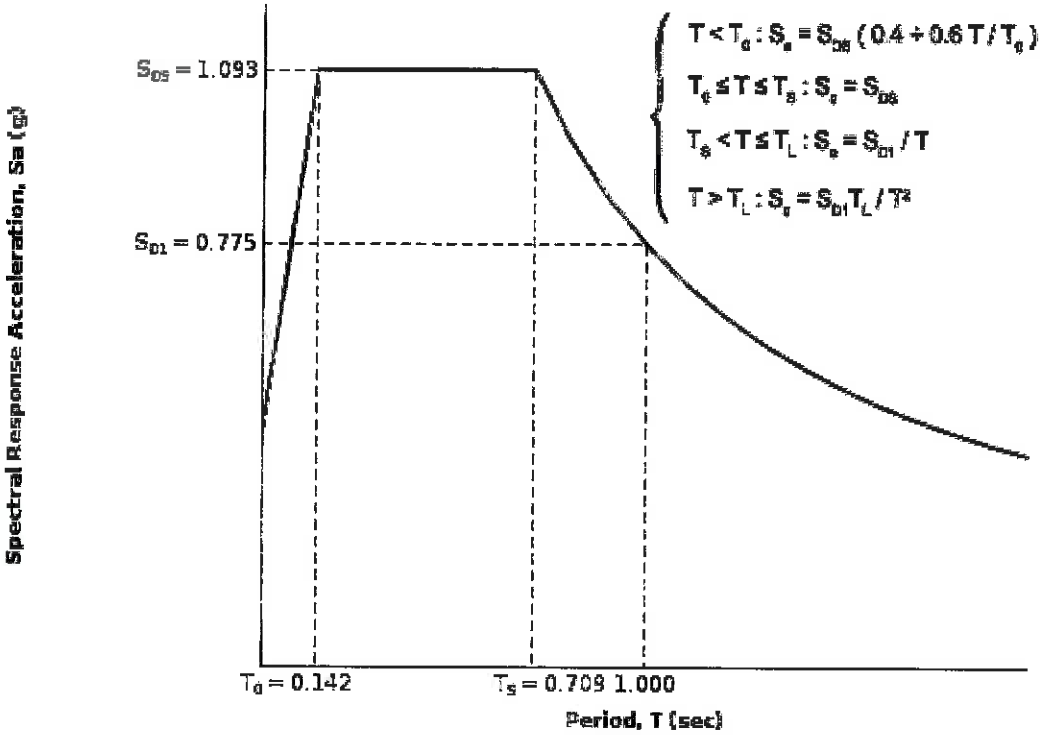
Section 11.4.5 — Design Response Spectrum

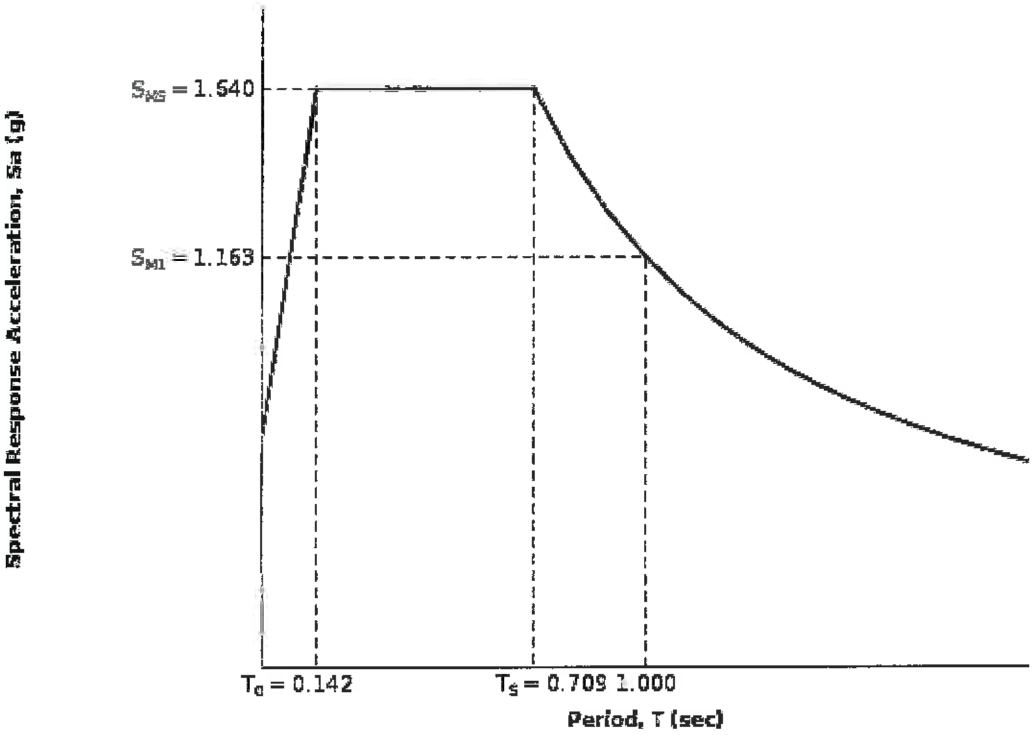
From Figure 22-12 [3] $T_L = 12$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.658

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.658 = 0.658$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.658 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.002

From Figure 22-18 [6] C_{R1} = 0.957

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	**RISK CATEGORY**		
	I or II	**III**	**IV**
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.093 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	**RISK CATEGORY**		
	I or II	**III**	**IV**
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.775 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

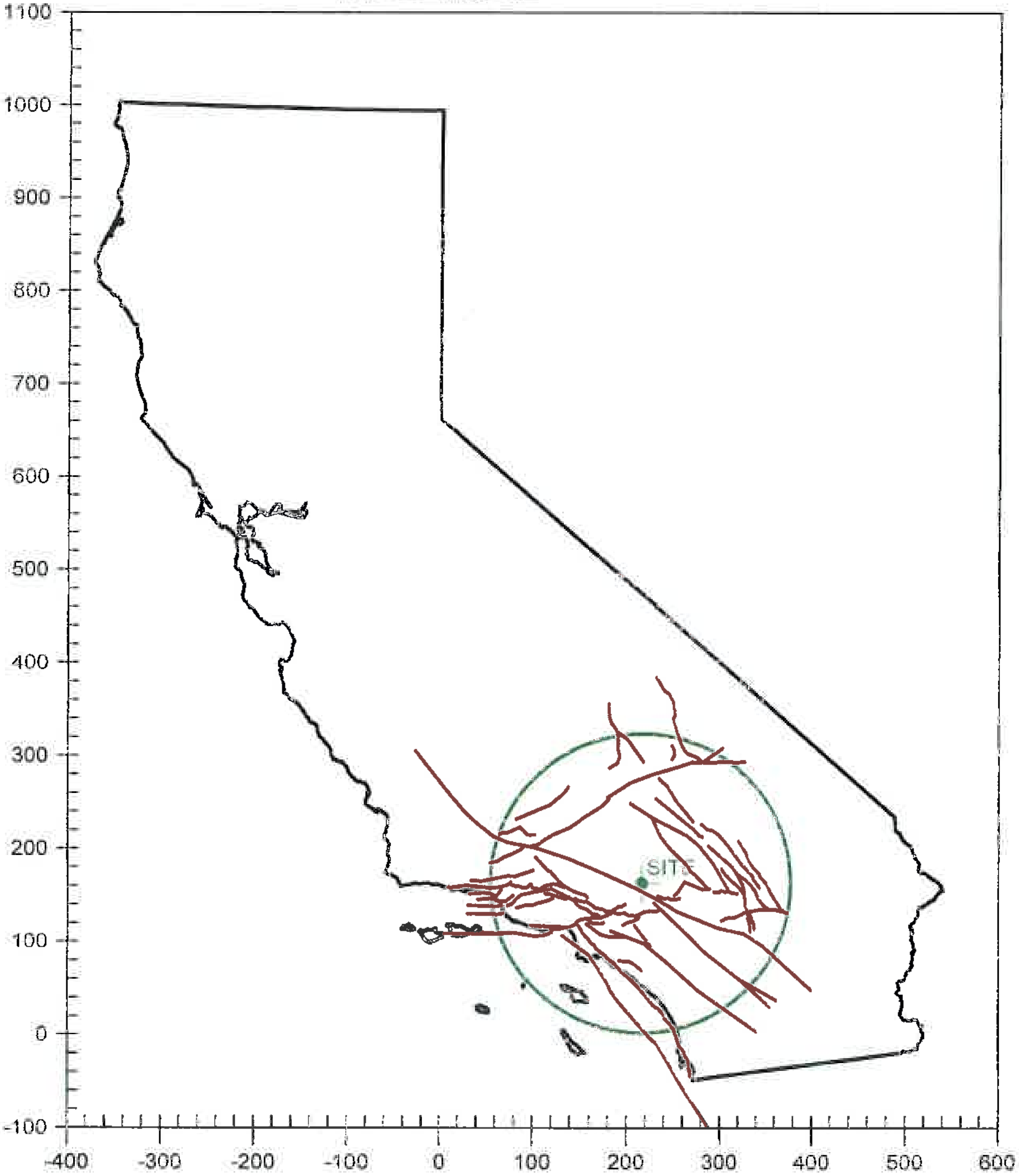
Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

CALIFORNIA FAULT MAP

15400 Dollar General



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                    *                     *
                    *    E Q F A U L T    *
                    *                     *
                    *    Version 3.00     *
                    *                     *
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                  DETERMINISTIC ESTIMATION OF
             PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 15400
                                                DATE: 05-21-2015

JOB NAME: 15400 Dollar General

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CGSFLTE.DAT


SITE COORDINATES:
   SITE LATITUDE:  34.4413
   SITE LONGITUDE:  117.6454

SEARCH RADIUS:   100  mi

ATTENUATION RELATION:   3) Boore et al. (1997) Horiz. - NEHRP D (250)
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   DISTANCE MEASURE:  cd_2drp
   SCOND:   0
   Basement Depth:  5.00 km     Campbell SSR:        Campbell SHR:
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED:  CGSFLTE.DAT


MINIMUM DEPTH VALUE (km):  0.0
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                              EQFAULT SUMMARY
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                        DETERMINISTIC SITE PARAMETERS
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-------------------------------------------------------------------------------
                                |             |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE |-------------------------------
          ABBREVIATED           |  DISTANCE   | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |  mi   (km)  |EARTHQUAKE|   SITE   |INTENSITY
                                |             | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|=============|==========|==========|=========
SAN ANDREAS - Mojave M-1c-3     |  5.1(  8.2) |   7.4    |  0.486   |    X
SAN ANDREAS - Whole M-1a        |  5.1(  8.2) |   8.0    |  0.667   |   XI
SAN ANDREAS - Cho-Moj M-1b-1    |  5.1(  8.2) |   7.8    |  0.600   |    X
SAN ANDREAS - 1857 Rupture M-2a |  5.1(  8.2) |   7.8    |  0.600   |    X
CUCAMONGA                       | 10.4( 16.7) |   6.9    |  0.291   |   IX
SAN ANDREAS - SB-Coach. M-2b    | 13.4( 21.6) |   7.7    |  0.303   |   IX
SAN ANDREAS - San Bernardino M-1| 13.4( 21.6) |   7.5    |  0.273   |   IX
SAN ANDREAS - SB-Coach. M-1b-2  | 13.4( 21.6) |   7.7    |  0.303   |   IX
CLEGHORN                        | 13.7( 22.0) |   6.5    |  0.159   |  VIII
SIERRA MADRE                    | 14.4( 23.2) |   7.2    |  0.269   |   IX
SAN JACINTO-SAN BERNARDINO      | 15.7( 25.2) |   6.7    |  0.160   |  VIII
CLAMSHELL-SAWPIT                | 16.8( 27.1) |   6.5    |  0.166   |  VIII
SAN JOSE                        | 21.4( 34.4) |   6.4    |  0.131   |  VIII
NORTH FRONTAL FAULT ZONE (West) | 23.1( 37.2) |   7.2    |  0.189   |  VIII
RAYMOND                         | 26.4( 42.5) |   6.5    |  0.118   |  VII
PUENTE HILLS BLIND THRUST       | 27.8( 44.7) |   7.1    |  0.155   |  VIII
VERDUGO                         | 28.5( 45.9) |   6.9    |  0.137   |  VIII
CHINO-CENTRAL AVE. (Elsinore)   | 28.8( 46.3) |   6.7    |  0.123   |  VII
HELENDALE - S. LOCKHARDT        | 29.6( 47.7) |   7.3    |  0.135   |  VIII
UPPER ELYSIAN PARK BLIND THRUST | 34.3( 55.2) |   6.4    |  0.091   |  VII
WHITTIER                        | 34.7( 55.9) |   6.8    |  0.092   |  VII
SIERRA MADRE (San Fernando)     | 37.0( 59.6) |   6.7    |  0.101   |  VII
SAN GABRIEL                     | 37.2( 59.8) |   7.2    |  0.108   |  VII
SAN JACINTO-SAN JACINTO VALLEY  | 37.6( 60.5) |   6.9    |  0.091   |  VII
HOLLYWOOD                       | 38.8( 62.5) |   6.4    |  0.083   |  VII
ELSINORE (GLEN IVY)             | 40.6( 65.3) |   6.8    |  0.081   |  VII
LENWOOD-LOCKHART-OLD WOMAN SPRGS| 42.1( 67.8) |   7.5    |  0.114   |  VII
NORTHRIDGE (E. Oak Ridge)       | 44.9( 72.2) |   7.0    |  0.102   |  VII
SANTA SUSANA                    | 47.8( 77.0) |   6.7    |  0.083   |  VII
NORTH FRONTAL FAULT ZONE (East) | 48.7( 78.4) |   6.7    |  0.082   |  VII
GRAVEL HILLS - HARPER LAKE      | 49.8( 80.1) |   7.1    |  0.081   |  VII
NEWPORT-INGLEWOOD (L.A.Basin)   | 50.5( 81.3) |   7.1    |  0.081   |  VII
SANTA MONICA                    | 50.9( 81.9) |   6.6    |  0.075   |  VII
LANDERS                         | 51.1( 82.3) |   7.3    |  0.089   |  VII
HOLSER                          | 51.4( 82.8) |   6.5    |  0.070   |  VI
SAN ANDREAS - Carrizo M-1c-2    | 52.3( 84.2) |   7.4    |  0.092   |  VII
SAN JOAQUIN HILLS               | 53.0( 85.3) |   6.6    |  0.073   |  VII
BLACKWATER                      | 53.9( 86.7) |   7.1    |  0.077   |  VII
JOHNSON VALLEY (Northern)       | 54.6( 87.8) |   6.7    |  0.061   |  VI
CALICO - HIDALGO                | 56.2( 90.5) |   7.3    |  0.082   |  VII


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                        DETERMINISTIC SITE PARAMETERS
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                                |  APPROXIMATE  |ESTIMATED MAX. EARTHQUAKE EVENT
                                |   DISTANCE    |--------------------------------
        ABBREVIATED             |   mi    (km)  | MAXIMUM  |   PEAK   |EST. SITE
        FAULT  NAME             |               |EARTHQUAKE|   SITE   |INTENSITY
                                |               | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|===============|==========|==========|=========
MALIBU COAST                    | 56.7(  91.3)  |   6.7    |  0.073   |  VII
ELSINORE (TEMECULA)             | 57.7(  92.9)  |   6.8    |  0.062   |  VI
GARLOCK (West)                  | 58.0(  93.4)  |   7.3    |  0.080   |  VII
PINTO MOUNTAIN                  | 59.1(  95.1)  |   7.2    |  0.075   |  VII
PALOS VERDES                    | 59.5(  95.7)  |   7.3    |  0.079   |  VII
SIMI-SANTA ROSA                 | 59.7(  96.1)  |   7.0    |  0.082   |  VII
NEWPORT-INGLEWOOD (Offshore)    | 60.7(  97.7)  |   7.1    |  0.070   |  VI
OAK RIDGE (Onshore)             | 61.5(  98.9)  |   7.0    |  0.080   |  VII
GARLOCK (East)                  | 62.1( 99.9 )  |   7.5    |  0.085   |  VII
EMERSON So. - COPPER MTN.       | 63.3( 101.9)  |   7.0    |  0.064   |  VI
SAN CAYETANO                    | 63.6( 102.4)  |   7.0    |  0.078   |  VII
SAN JACINTO-ANZA                | 63.8( 102.7)  |   7.2    |  0.071   |  VI
ANACAPA-DUME                    | 67.2( 108.2)  |   7.5    |  0.097   |  VII
PISGAH-BULLION MTN.-MESQUITE LK | 71.0( 114.3)  |   7.3    |  0.069   |  VI
SANTA YNEZ (East)               | 71.9( 115.7)  |   7.1    |  0.061   |  VI
WHITE WOLF                      | 73.4( 118.2)  |   7.3    |  0.081   |  VII
BURNT MTN.                      | 73.6( 118.5)  |   6.5    |  0.044   |  VI
EUREKA PEAK                     | 74.6( 120.1)  |   6.4    |  0.041   |   V
PLEITO THRUST                   | 75.0( 120.7)  |   7.0    |  0.068   |  VI
So. SIERRA NEVADA               | 75.9( 122.2)  |   7.3    |  0.079   |  VII
SAN ANDREAS - Coachella M-1c-5  | 76.0( 122.3)  |   7.2    |  0.062   |  VI
LITTLE LAKE                     | 81.5( 131.2)  |   6.9    |  0.050   |  VI
ELSINORE (JULIAN)               | 81.9( 131.8)  |   7.1    |  0.055   |  VI
BIG PINE                        | 81.9( 131.8)  |   6.9    |  0.050   |  VI
CORONADO BANK                   | 82.6( 132.9)  |   7.6    |  0.072   |  VI
VENTURA - PITAS POINT           | 85.5( 137.6)  |   6.9    |  0.059   |  VI
TANK CANYON                     | 85.9( 138.2)  |   6.4    |  0.045   |  VI
M.RIDGE-ARROYO PARIDA-SANTA ANA | 86.7( 139.5)  |   7.2    |  0.068   |  VI
PANAMINT VALLEY                 | 91.3( 146.9)  |   7.4    |  0.060   |  VI
OWL LAKE                        | 91.7( 147.5)  |   6.5    |  0.037   |   V
ROSE CANYON                     | 92.0( 148.1)  |   7.2    |  0.053   |  VI
CHANNEL IS. THRUST (Eastern)    | 93.0( 149.6)  |   7.5    |  0.075   |  VII
RED MOUNTAIN                    | 93.2( 150.0)  |   7.0    |  0.058   |  VI
OAK RIDGE(Blind Thrust Offshore)| 93.8( 150.9)  |   7.1    |  0.061   |  VI
OAK RIDGE MID-CHANNEL STRUCTURE | 93.9( 151.1)  |   6.6    |  0.047   |  VI
SAN JACINTO-COYOTE CREEK        | 94.0( 151.2)  |   6.6    |  0.038   |   V
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-END OF SEARCH-   76 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SAN ANDREAS - Whole M-1a          FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 5.1 MILES (8.2 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.6666 g
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